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March 1995

US Army Corps
of Engineers
Waterways Experiment
Station

Physical Model Study of Revere Beach, Massachusetts

by Donald L. Ward

WES

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Prepared for U.S. Army Engineer Division, New England

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Waterways Experiment Station
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Final report

Prepared for U.S. Army Engineer Division, New England
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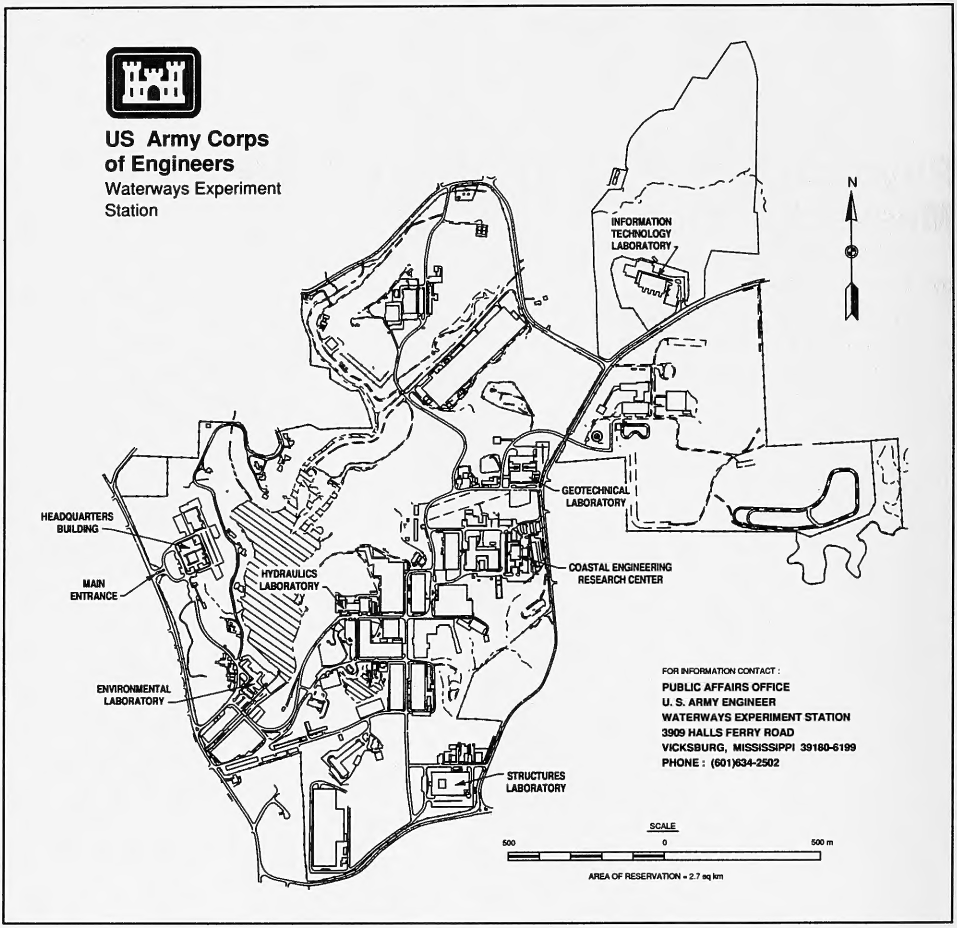

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Preface

The physical model study of Revere Beach, Massachusetts, reported herein was requested by U.S. Army Engineer Division, New England (CENED), as part of the Saugus River and Tributaries Flood Damage Reduction Project. The investigation was conducted at the Coastal Engineering Research Center (CERC) of the U.S. Army Engineer Waterways Experiment Station (WES) between December 1992 and July 1993. The physical model study was intended to provide overtopping data and empirical equations that could be incorporated into numerical model coastal processes, to aid in the design of a new dike to provide flood protection for the city of Revere, and to supplement a coastal processes study conducted by personnel in the Coastal Oceanography Branch and Coastal Processes Branch of the Research Division at CERC (Smith et al. 1994).

This study was conducted by personnel of CERC under the general direction of Dr. James R. Houston, Director, CERC, and Mr. Charles C. Calhoun, Jr., Assistant Director, CERC. Direct supervision was provided by Messrs. C. E. Chatham, Chief, Wave Dynamics Division (WDD), and D. D. Davidson, Chief, Wave Research Branch (WRB), WDD. This report was prepared by Messrs. Donald L. Ward, Principal Investigator, WEB, and John M. Heggins, Computer Technician, WRB.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
acre-feet	1,233.489	cubic meters
cubic feet per second	0.02831685	cubic meters per second
cubic yards	0.7645549	cubic meters
feet	0.3048	meters
inches	2.54	centimeters
miles	1.609347	kilometers
nautical miles	1.852	kilometers
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter

1 Introduction

The Prototype

Revere Beach in the City of Revere, Massachusetts, is located on the Massachusetts coastline approximately 6 miles[1] northeast of Boston (Figure 1). The beach is located on Broad Sound, bordered by Roughans Point headland to the south and Point of Pines to the north, and is partially sheltered by the Nahant Peninsula to the northeast. Revere Beach is the oldest public beach in the nation, with boundaries established in 1895.

Erosion of the beach led to construction of protective seawalls in the 1920's along most of the reach. The seawalls were not sufficient to prevent severe flooding of backshore areas. The beach continued to erode, and waterfront establishments suffered from the floodings.

Approximately 600,000 cu yd of fill were placed along Revere Beach in 1991 as part of the Revere Beach Restoration project constructed by the U.S. Army Engineer Division, New England (NED). The fill placed a 50-ft-wide berm in front of the seawall at elevation 18.0 ft mean low water (mlw), which is typically 2 to 3 ft below the seawall crest. Although the beach was not designed or justified for flood level reduction, it does reduce wave overtopping, resulting in incidental flood reduction benefits.

Eight bathymetric profiles were surveyed along Revere Beach and Point of Pines after a 1978 storm, extending offshore from the seawall for approximately 10,000 ft (Figure 2). The surveys were repeated after a beachfill project in 1991, and a storm which followed in October 1991.

[1] A table of factors for converting non-SI units of measurement to SI units is presented on page vii.

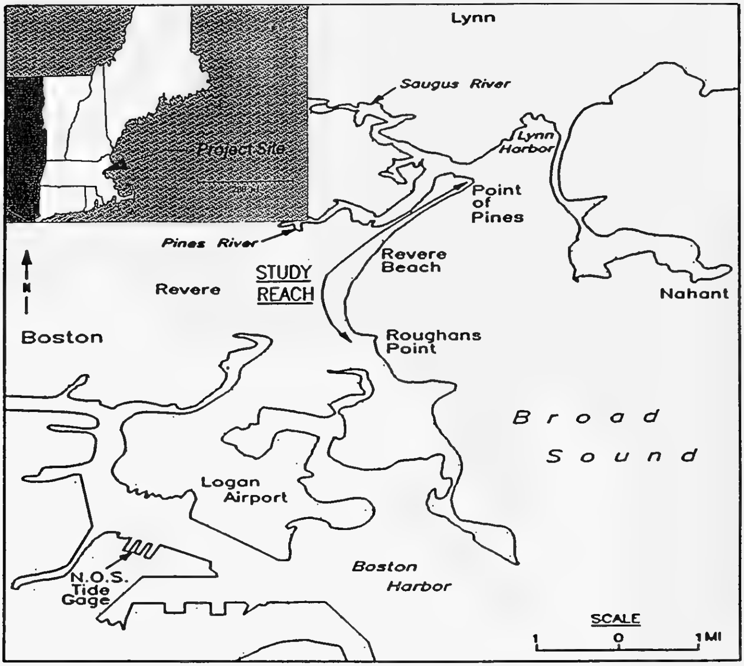

Figure 1. Location map

The Problem

As part of the Saugus River and Tributaries Flood Damage Reduction Project, a flood control plan includes construction of a tidal floodgate across the mouth of the Saugus River, and new walls, dikes, revetments, and dunes along the shorefronts of Revere and Lynn. Also, land is being purchased along the Saugus and Pines Rivers to provide a holding area for any floodwater that may overtop the beach and seawall as well as retaining rainfall runoff during floodgate closure. The problem was to determine the amount of overtopping expected, to assist NED in determining the quantity of land to be purchased. Wave hindcast studies were conducted by the Research Division of the U.S. Army Engineer Waterways Experiment Station's Coastal Engineering Research Center (CERC) to determine major storm conditions at offshore locations in the area, and numerical models were used to propagate storm waves shoreward to the beach. The numerical models could not adequately predict overtopping along the beach, and CERC's Wave Dynamics Division was requested to determine the overtopping rates through physical model studies, using wave and water level data supplied by the numerical models. Information on the physical model studies is contained in this report. For information on the numerical model studies, see Smith et al. (1994).

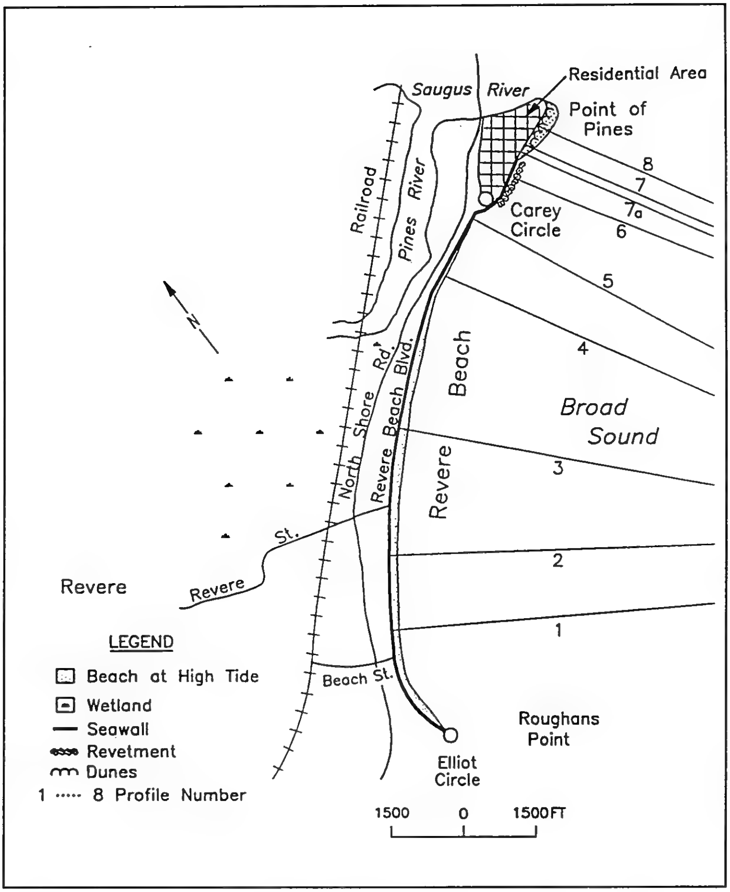

Figure 2. Location of profiles used in SBEACH simulations

To further protect against flood damage, a dike has been proposed for construction on the west side of Revere Boulevard along a reach of Revere Beach, and the state is interested in developing the dike into public parkland. Overtopping values are needed to design the dike, but could not be accurately estimated by analytical techniques due to complexities of wave action flowing over the beach bathymetry, overtopping the seawall, crossing Revere Boulevard, overtopping a toe wall at the seaward edge of the dike, and then flowing up the “park” dike. CERC was therefore asked to determine overtopping rates for the proposed park dike through physical model tests. To assist NED in determining the level of non-federal funding, CERC also was asked to determine a minimum dike configuration, rubble-mound structure for flood control only, that would provide protection against overtopping along Revere Boulevard.

Scope of Work

The physical modeling study was divided into four tasks. Task A was to confirm the validity of numerical and physical models by recreating a known storm event and comparing the overtopping in the physical model to measured overtopping in the backshore, using the beach profile that existed at that time. Task B was to determine overtopping for the duration of the selected design storm event along Revere Beach using beach profiles surveyed after the beachfill project. Data from Task B also were used by CERC's Research Division to develop a "bore runup overtopping module" for Revere Beach to be used with numerical models. Task C was to use one of the profiles taken before the beachfill project and determine overtopping rates for storm conditions selected from a synthetic storm database developed from major storm events identified by wave hindcasting. Data from Task C were used by CERC's Research Division to develop a "broken wave overtopping module" for Revere Beach to be used with numerical models.

The fourth task of the physical modeling effort was the study of the proposed dike along Revere Boulevard. The study included determination of overtopping rates for the proposed park dike and a rubble-mound dike for flood control only when fronted by profiles from both the 1978 and 1991 surveys and subjected to severe storm conditions.

2 Test Facility

Two-dimensional (2-D) physical model tests were conducted in CERC's 150-ft-long by 1.5-ft-wide by 3.0-ft-deep wave tank ("18-in. flume") and 150-ft-long by 3.0-ft-wide by 3.0-ft-deep wave tank ("3-ft flume"). In both flumes, waves were generated by a piston-type wave board powered by an electro-hydraulic pump controlled by a computer-generated signal. The 18-in. flume has an existing 1:30 (V:H) concrete slope starting 60 ft from the wave board; the 3-ft flume has a 1:20 concrete slope starting 36 ft from the wave board and extending for 10 ft, followed by an approximately 1:100 slope. Pre-test conditons of the 18-in. and 3-ft flumes are illustrated in Figures 3 and 4, respectively. The flumes were modified for each test to meet specific profile needs.

The models were built to a geometrically undistorted linear scale of 1:20 (model:prototype) for Task A and 1:30 for Tasks B, C, and the dike study. Based on Froude's model law (Stevens et al. 1942), the following model-to-prototype relationships were derived. Dimensions are in terms of length *L* and time *T*.

Characteristic	Dimension	Model-to-Prototype Scale Relation		
			1:20	1:30
Length	L	L_r	1:20	1:30
Area	L^2	$A_r = (L_r)^2$	1:400	1:900
Volume	L^3	$V_r = (L_r)^3$	1:8,000	1:27,000
Time	T	$T_r = (L_r)^{1/2}$	1:4.472	1:5.477

Water that overtopped the seawalls during physical model tests was pumped into a rectangular catch basin at the conclusion of the test run. The change in elevation of water in the catch basin was then measured with a point gauge and converted to prototype overtopping rate in cubic feet per second per linear foot of prototype seawall (cfs/ft) by the following relationship:

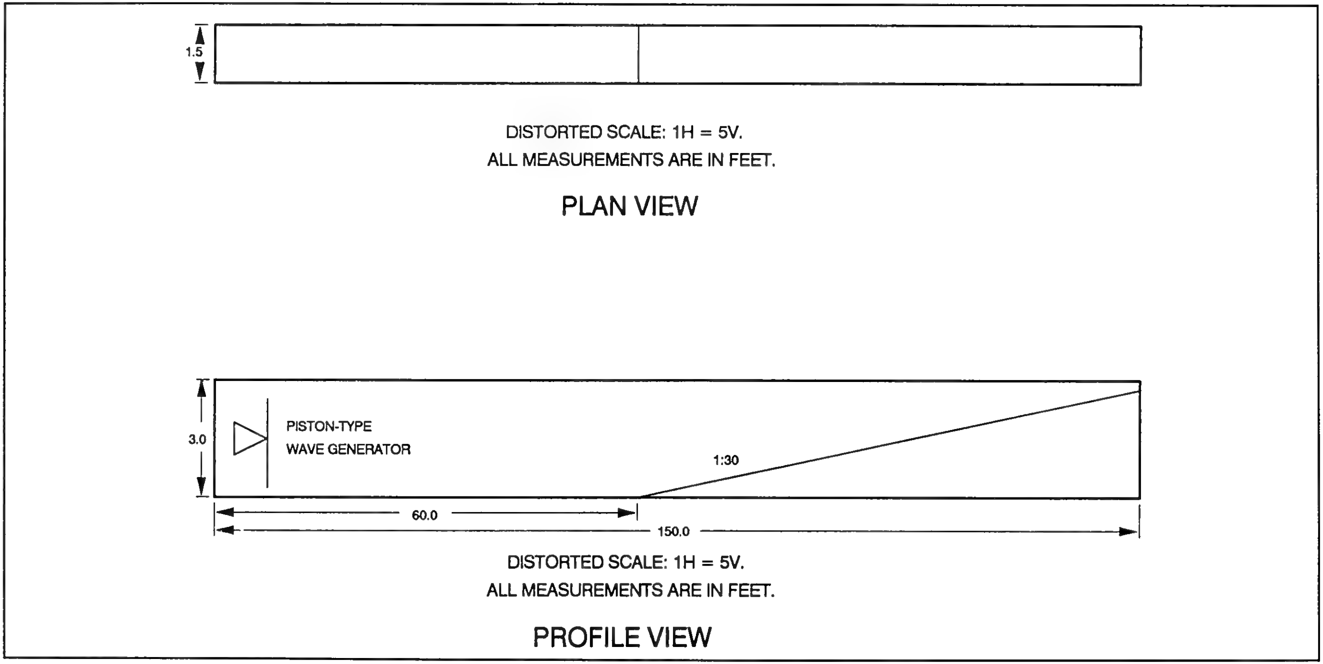

Figure 3. Plan and profile views of 18-in. wave flume

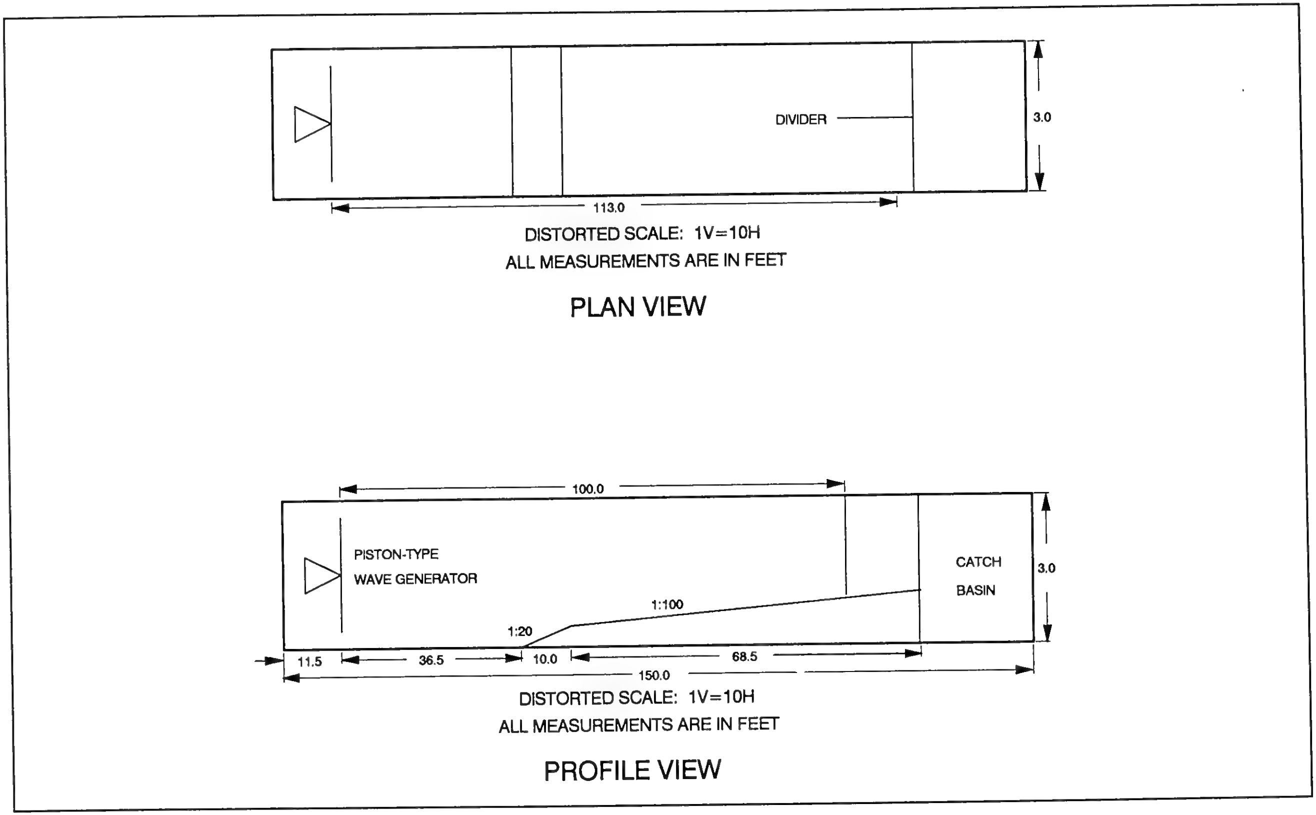

Figure 4. Plan and profile views of 3-ft wave flume

$$OT = \frac{\left(A_{cb} \times S^2\right) \times \left(H_{cb} \times S\right)}{\left(T_{run} \times S^{1/2}\right) \times \left(W_{flume} \times S\right)}$$

where

OT = prototype overtopping rate in cfs/ft

A_{cb} = cross-sectional area of model catch basin, ft^2

S = scale factor, either 20 for 1:20 or 30 for 1:30

H_{cb} = change in water surface elevation in the model catch basin, ft

T_{run} = time of model test run, sec

W_{flume} = width of model test flume, ft

For each set of tests, the cross-sectional area of the model catch basin, flume width, time of model run, and scale factor were constants. The overtopping rate could therefore be calculated as

$$OT = \frac{A_{cb} \times S^{3/2}}{T_{run} \times W_{flume}} \times H_{cb}$$

where the first term is a constant and the only variable is change in elevation of the water surface in the catch basin.

3 Research Tasks A, B, and C

Task A

Purpose

The purpose of Task A was to validate the numerical and physical models by reproducing the effects of a known storm event. The storm selected for the test occurred in February, 1978; surveys of high-water marks in ponding areas provided an estimate of the total overtopping over the reach represented by Profile 2, which is located fronting the proposed park dike. Using wave data supplied by CERC's Research Division, Task A simulated the 1978 storm in the physical model to determine if overtopping measured in the 2-D model corresponded to the estimated prototype overtopping volume.

Selection of test conditions

The National Ocean Survey (NOS) Boston Harbor tide gauge provided data on local water levels. Wave conditions were determined by CERC's Research Division with the following numerical models: deepwater wave hindcast information was brought shoreward using numerical model SHALWV, diffracted and refracted into Broad Sound by numerical model REF/DIF, and transformed shoreward along each of the surveyed profiles using numerical model SBEACH. SHALWV uses spectral wave information; REF/DIF and SBEACH use monochromatic waves. The height of the average of the one-third highest waves (significant wave height H_s) and the period of peak energy density (T_p) calculated from the wave energy spectrum determined by SHALWV were chosen to characterize the monochromatic wave transformation in REF/DIF and SBEACH.

Still-water level (swl), wave height, and wave period were determined for each hour during the selected storm to define the storm profile, and test conditions for the physical model study were selected from the storm profile. Figure 5 shows the storm profile used as input to SBEACH. For the physical model study, water level at the peak of the second tide cycle

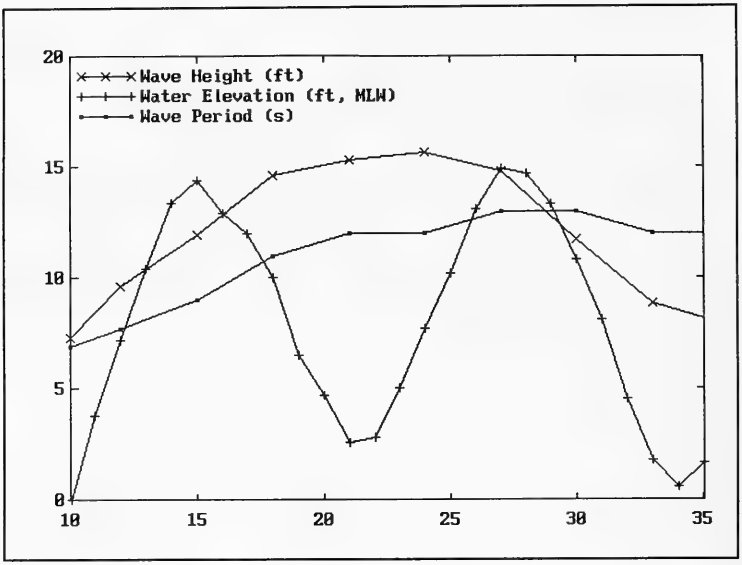

Figure 5. Storm profile for storm of February 1978

(hr 27) was selected to test the worst recorded conditions, a low water level was selected to allow estimation of the period of the storm in which overtopping could be neglected (approximately hr 13, 18, 25, and 30), and an intermediate water level was selected to represent the rest of the storm (approximately hr 14, 16, 26, and 29). Each of the lower water levels selected was tested with wave conditions on the incoming and outgoing tides of both tide cycles. Storm hour, swl, wave height, and wave period are shown in Table 1 after shoaling in SBEACH to the approximate location of the wave generator in the physical model (approximately 2,000 ft offshore).

Because of the amount of time involved in changing water levels in the wave flume, it was desired to use a constant water level for each of the four points on each of the two lower water levels. The swl at hr 25 was chosen for the lowest water level (10.8 ft mean low water (mlw)) and the swl at hr 16 was chosen for the higher water level (13.0 ft mlw). Linear interpolation based on water level and two surrounding data points was used to adjust wave conditions to the selected points in the storm profile. For example, the swl at hr 13 was 10.5 ft mlw, and 13.5 ft at hr 14. Linear interpolation determined that the test conditions of 10.8 ft mlw and 13.0 ft mlw occurred at hr 13.27 and hr 13.83, respectively. Using the same interpolation for wave height and wave period yielded the interpolated results shown in Table 1.

Physical model tests were conducted with irregular waves following the Texel Marsen Arsloe (TMA) spectrum (Hughes 1984), which is a

Table 1
Wave Data from Numerical Model SBEACH and Interpolated Wave Conditions Tested for 1978 Storm at Revere Beach, Profile 2

Hour	SWL ft, mlw	Wave Height ft	Wave Period sec	SWL Tested ft, mlw	Interp Wave Ht ft	Interp Wave Per sec	Overtopping Rate, cfs/ft
13	10.5	6.0	8.1	10.8	6.1	8.1	0.0066
14	13.5	7.2	8.6	13.0	7.0	8.6	0.0643
15	14.2	10.8	9.0				
16	13.0	10.1	9.7	13.0	10.1	9.7	0.1004
17	13.0	6.8	10.3				
18	11.2	6.1	11.0	10.8	6.0	11.0	0.008
19	8.0	5.0	11.3				
24	9.0	5.3	12.0				
25	10.8	6.0	12.3	10.8	6.0	12.3	0.0077
26	13.6	7.0	12.7	13.0	6.8	12.6	0.0843
27	15.4	7.7	13.0	15.4	7.7	13.0	1.3553
28	15.5	7.7	13.0				
29	14.2	7.2	13.0	13.0	6.8	13.0	0.959
30	11.5	6.3	13.0	10.8	6.0	12.9	0.0063
31	8.8	5.3	12.7				

shallow-water modification of the Joint North Sea Wave Project (JONSWAP) spectrum (Hasselman et al. 1973). Monochromatic wave heights and periods were obtained from SBEACH at a distance offshore corresponding to the approximate location of the wave generator in the physical model study and used as the height of the zeroth moment (H_{mo}) and T_p for the wave spectra.

Physical model tests were conducted for 30 min for each of the four test conditions at each of the two lower water levels (10.8 and 13.0 ft mlw). Due to the small amount of overtopping at these water levels, the water level in the flume did not decrease appreciably during the tests. During tests at the highest water level (hr 27), test runs were limited to 2 min each to allow the overtopped water to be added back into the flume to maintain the desired water level. Ten 2-min runs were conducted at the highest water level. Different wave signals were generated for each 2-min run.

After completing the test series, overtopping volume collected during the tests was found to be much higher than expected. Examination of the input data showed that water levels provided by SBEACH included computer estimates of wave setup. However, wave setup occurs naturally in a wave flume and is not input with the still-water level. The test series was therefore rerun with information from SBEACH by excluding the wave setup adjustment. Wave conditions and water levels for the second set of tests are shown in Table 2. Note that hr 15 and 28 were added to the second set of tests to more accurately reflect the storm profile. Results of only the second set of tests were used to calculate overtopping during the storm, but results of both sets of tests were used in determining a regression equation relating water level, wave conditions, and freeboard to overtopping rate.

Table 2
Revised Wave Data from Numerical Model SBEACH and Interpolated Wave Conditions Tested for 1978 Storm at Revere Beach, Profile 2

Hour	SWL ft, mlw	Wave Height ft	Wave Period sec	SWL Tested ft, mlw	Interp Wave Ht ft	Interp Wave Per sec
13	10.1	6.0	8.1	11.0	6.3	8.2
14	13.2	7.2	8.6	13.2	7.2	8.6
15	14.3	10.8	9.0	14.3	10.8	9.0
16	13.0	10.1	9.7	13.2	10.2	9.6
17	12.1	6.8	10.3			
18	10.2	6.1	11.0	11.0	6.4	10.5
19	8.0	5.0	11.3			
24	7.5	5.3	12.0			
25	10.0	6.0	12.3	11.0	6.3	12.4
26	12.9	7.0	12.7	13.2	7.1	12.7
27	14.8	7.7	13.0	14.8	7.7	13.0
28	14.7	7.7	13.0	14.8	7.7	13.0
29	13.4	7.2	13.0	13.2	7.1	13.0
30	11.0	6.3	13.0	11.0	6.3	13.0
31	8.1	5.3	12.7			

For the second set of tests, the swl at hr 30 was chosen for the lowest water level (11.0 ft mlw), and the swl at hr 14 was chosen for the higher water level (13.2 ft mlw). Similar to the first set of tests, linear interpolation was used to determine the time at which the swl to be tested occurred and the wave height and period at that time. Test conditions are shown in Table 2.

Because of time restraints imposed by having to rerun the storm profile, the second set of tests was reduced to a single 20-min run at each of the four test conditions on the lowest water level (11.0 ft mlw), two 10-min runs at each of the test conditions at the next higher water level (13.2 ft mlw), and five 2-min runs at each of the three highest water levels. As in the earlier set of tests, multiple runs of short duration were used at the highest water levels to allow the overtopped water to be returned to the flume to maintain the swl.

Determination of model profile

The existing 1:30 concrete slope in the 18-in flume did not match the beach survey taken after the 1978 storm. Therefore, an entirely new profile was constructed and installed seaward of the existing concrete slope.

The beach profile was displayed on a computer screen and an idealized profile was determined by matching a series of straight lines to the actual profile as closely as feasible, including a horizontal line to use as the flume bottom. The actual profile and the idealized profile are shown in Figure 6.

With the depth at the flume bottom determined, model scale was established by limitations of the wave generator. The wave generator was unable to generate the required signals at scales larger than 1:20; therefore, the model was constructed at a 1:20 scale.

The idealized profile was constructed of plywood and placed in the wave flume over the concrete slope. When the slope was within 0.75 in. of the flume bottom (thickness of the plywood), 20-gauge sheet metal was used to extend the slope to the bottom of the flume. A vertical seawall was placed at the top of the plywood slope. Water overtopping the seawall accumulated behind the seawall and was pumped into a separate canister for accurate measurement of the overtopping volume at the end of each test run.

Results

Overtopping rates for the first set of conditions tested are listed in Table 1; overtopping rates for the repeated storm profile are listed in Table 3. Data in the tables have been converted to prototype scale.

To determine total overtopping during the storm, it was assumed that the overtopping rate determined for a given point in the storm profile was constant over the time period extending from half-way between the given point and the preceding point to half-way between the given point and the following point. Because data were available at every 1-hr interval of the storm, overtopping rates at the first and last points tested were assumed to exist for 1/2 hr before and after the point tested, respectively. Multiplying the overtopping rate for a tested point in the storm profile by the length of time the storm was assumed constant at those conditions yielded the volume of overtopping for that test per foot of seawall, and multiplying by the length of seawall contributing to the flood zone yielded the total volume

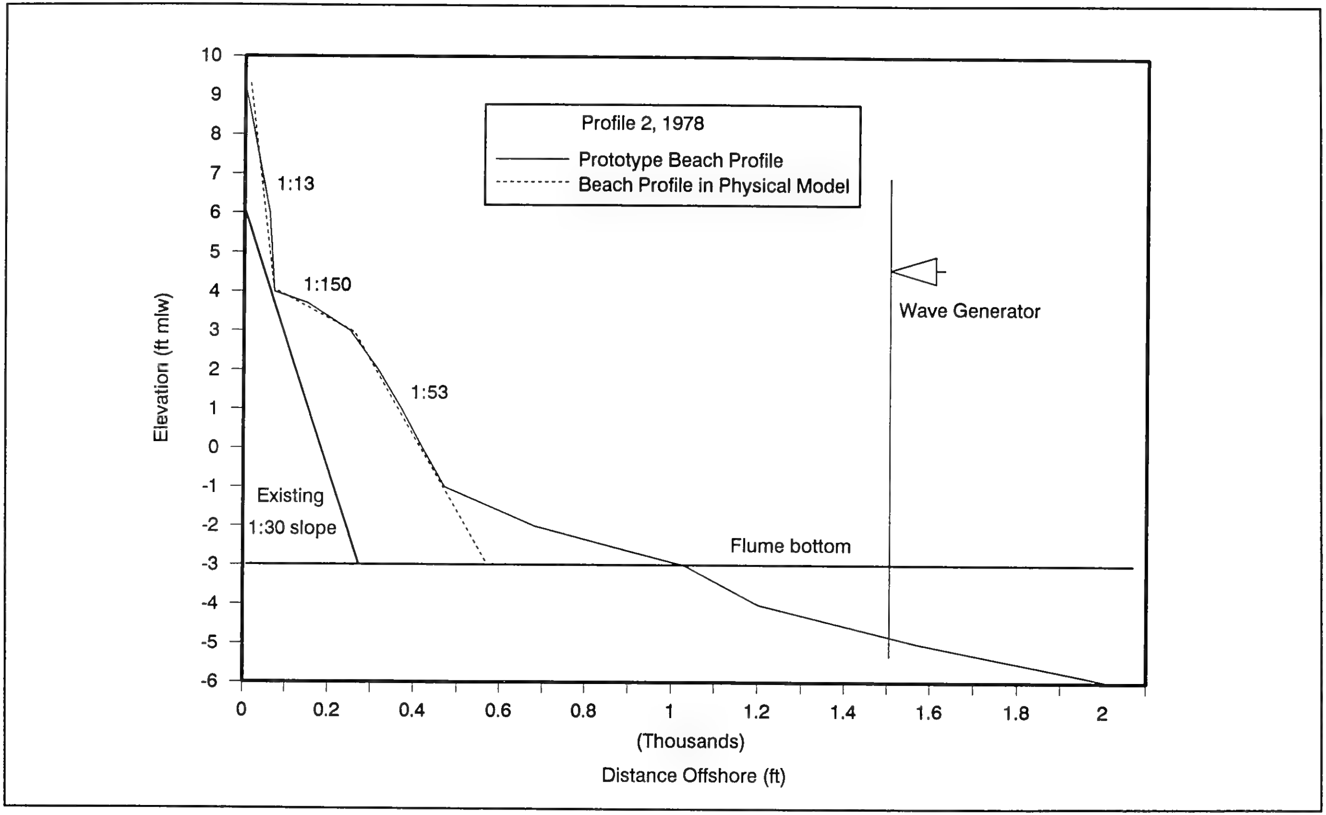

Figure 6. Profile 2 as surveyed in 1978 and as reproduced for physical model study

Table 3
Overtopping for 1978 Storm at Profile 2 for Prototype Seawall Length of 3,890 ft

Hour	Interp Hour	Begin Hour	End Hour	Total Time sec	Overtopping Rate, cfs/ft	Acre-ft for 3,890-ft Length
13	13.29	12.50	13.64	4,094	0.0097	3.56
14	13.98	13.64	14.49	3,077	0.0994	27.32
15	15.00	14.49	15.44	3,421	0.5200	158.88
16	15.88	15.44	16.73	4,642	0.1659	68.76
17						
18	17.58	16.73	18.50	6,366	0.0103	5.86
25	25.34	24.50	25.74	4,458	0.0100	3.97
26	26.13	25.74	26.82	3,879	0.1674	57.99
27	27.50	26.82	27.50	2,463	0.8141	179.08
28	27.50	27.50	28.30	2,888	0.8141	209.93
29	29.10	28.30	29.55	4,500	0.1359	54.60
30	30.00	29.55	30.50	3,412	0.0116	3.54
					Storm Total	773 acre-ft

of overtopping over the seawall for the time period that was tested. For this series of tests, NED determined that 3,890 ft of seawall would contribute to the flood zone. Overtopping rates and volume for each hour of the storm are shown in Table 3.

Based on surveys of high-water marks, NED calculated that about 600 acre-ft of water overtopped the seawall during the 1978 storm. The physical model test showed a total overtopping of 773 acre-ft, roughly 29 percent higher than the surveys had indicated. Due to uncertainties in the surveyed results, numerical models, and physical model tests, test results were surprisingly close to the predicted results. Uncertainties in the tests are discussed in Chapter 4, "Discussion of Research Tasks A, B, and C."

Task B

Purposes

Purposes of Task B were to determine total overtopping for the design storm event for the beach profiles surveyed in 1991 (after the beachfill project) and to generate input data for a bore runup overtopping module to be used with numerical models by CERC's Research Division. Using wave data supplied by CERC's Research Division, Task B reproduced in physical models the five beach profiles located along Revere Beach and subjected them to the design storm event. Overtopping was measured for each profile at each hour of the storm tested.

Selection of test conditions

The design storm event, or Standard Project Northeaster (SPN), was based on a storm that occurred in November 1945. Wave conditions during the storm were obtained by hindcasting; still-water levels during the storm were obtained from the NOS Boston Harbor tide gauge. NED defined the SPN as the wave conditions from the storm profile determined by hindcasting for the November 1945 storm, but with an additional foot added to the swl recorded by the NOS tide gauge throughout the storm. The SPN was input by CERC's Research Division into the numerical models listed under Task A to obtain storm conditions at Revere Beach. Figure 7 shows the storm profile for the SPN used as input to SBEACH. Conditions to be tested in the physical model were selected from the storm profile to include the worst conditions that occurred during the storm (hr 30) plus conditions at two lower water levels during both tide cycles shown in the storm profile (hr 27, 33, 40, and 45 for the lowest water level and hr 28, 32, and 43 for the higher water level). However, the static beachfill profile reduced overtopping to such an extent that no overtopping occurred during tests at the lowest water level; therefore, additional points from the peaks of the tide cycles were selected for testing. As in Task A, linear interpolation was used where feasible to adjust wave heights and periods to maintain a constant swl for tests of the incoming and outgoing tides in both tide cycles. Test conditions and the approximate hour of the storm represented are listed in Table 4 after shoaling in SBEACH to the approximate location of the wave generator. As in the second set of tests in Task A wave setup was allowed to occur naturally in the wave flume, and the wave setup adjustment from SBEACH was not used.

Determination of model profile

Beach Profiles 1, 3, 4, and 5 were reproduced in the 18-in. flume at a geometrically undistorted scale of 1:30. Examination of beach surveys taken in 1991 indicated portions of the profiles could be represented by the existing 1:30 concrete slope in the wave flume and the flat bottom of

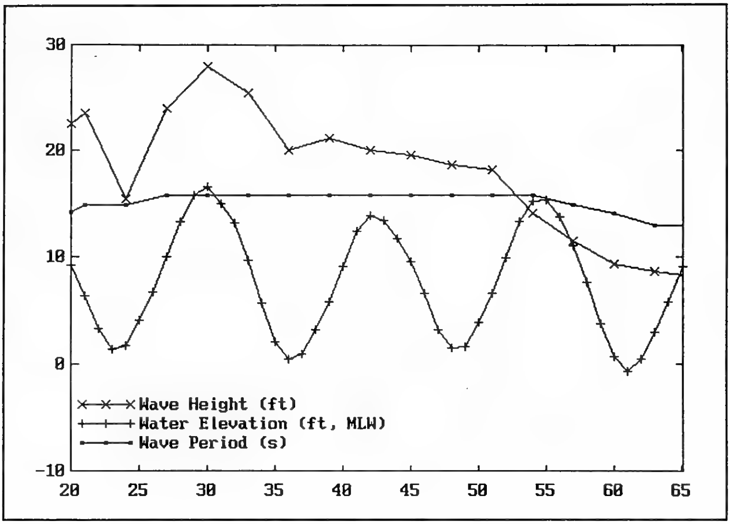

Figure 7. Storm profile for SPN from storm of November, 1945

the flume. Shoreward of the 1:30 slope, sheet metal was used to reproduce the steeper portion of the beach profile. A vertical seawall was placed at the top of the slope, and water overtopping the seawall was collected and measured to determine overtopping rates. Surveyed profiles and model representations of Profiles 1, 3, 4, and 5 are shown in Figures 8 through 11, respectively.

Seawall elevations varied over the reach represented by each profile. A representative seawall height was selected for each profile except Profile 1; two representative seawall heights were selected for Profile 1. Selected seawall elevations are listed in Table 5.

Beach surveys started at the foot of the seawall, and the elevation at the foot of the seawall was reproduced in all model profiles except Profile 1. The beach surveyed at Profile 1 measured an elevation of +21.0 ft mlw at the base of the seawall with a seawall crest elevation reported at +21.4 ft mlw, providing a freeboard of 0.4 ft. However, selected representative seawall elevations for that segment of Revere Beach were +19.8 ft and +20.7 ft, both of which are lower than the beach survey. Because the reaches represented by both seawall elevations were significant, it was decided to conduct the Profile 1 test series twice, with one complete set at a seawall elevation of +19.8 ft and one complete set at a seawall elevation of +20.7 ft. For the first set of tests on Profile 1, the profile was modeled such that the beach slope extended to an elevation of +19.4 ft and then remained at a constant elevation until reaching the seawall, resulting in a freeboard of 0.4 ft. For the second set of tests, the same slope was used to

Table 4
Wave Data from SBEACH and Interpolated Wave Conditions with 1991 Profiles at Revere Beach

Hour	SWL ft, mlw	Wave Height, ft	Wave Period, sec	SWL Tested ft, mlw	Interp Hour	Interp Wave Height, ft	Interp Wave Period, sec
Profile 5							
27	10.0	8.8	15.9	10.00	27.00	8.80	15.90
28	13.4	9.1	15.9	13.40	28.00	9.10	15.90
29	15.9	8.9	15.9	15.90	29.00	8.90	15.90
30	16.6	8.7	15.9	16.60	30.00	8.70	15.90
31	15.0	8.3	15.9	15.00	31.00	8.30	15.90
32	13.2	9.0	15.9	13.40	31.89	8.92	15.90
33	9.7	8.5	15.9	10.00	32.91	8.54	15.90
43	13.5	7.0	15.9	13.40	43.06	6.89	15.90
44	11.8	5.1	15.9				
45	9.6	4.6	15.9	10.00	44.82	4.69	15.90
Profile 4							
27	10.0	7.4	15.9	10.00	27.00	7.40	15.90
28	13.4	7.9	15.9	13.40	28.00	7.90	15.90
29	15.9	10.4	15.9	15.90	29.00	10.40	15.90
30	16.6	7.1	15.9	16.60	30.00	7.10	15.90
31	15.0	9.6	15.9	15.00	31.00	9.60	15.90
32	13.2	7.7	15.9	13.40	31.89	7.91	15.90
33	9.7	7.1	15.9	10.00	32.91	7.15	15.90
42	13.9	9.9	15.9	13.90	42.00	9.90	15.90
43	13.5	7.0	15.9	13.40	43.06	6.96	15.90
44	11.8	6.3	15.9				
45	9.6	7.5	15.9	10.00	44.82	7.28	15.90
Profile 3							
27	10.00	4.2	15.9	10.00	27.00	4.20	15.90
28	13.4	4.4	15.9	13.40	28.00	4.40	15.90
29	15.9	4.5	15.9	15.90	29.00	4.50	15.90
30	16.6	4.4	15.9	16.60	30.00	4.40	15.90
31	15.0	4.4	15.9				
32	13.2	4.4	15.9	13.40	31.89	4.40	15.90
33	9.7	4.1	15.9	10.00	32.91	4.13	15.90

(Continued)

Table 4 (Concluded)							
Hour	SWL ft, mlw	Wave Height, ft	Wave Period, sec	SWL Tested ft, mlw	Interp Hour	Interp Wave Height, ft	Interp Wave Period, sec
Profile 3 (Concluded)							
42	13.9	7.3	15.9	13.90	42.00	7.30	15.90
43	13.5	7.1	15.9	13.40	43.06	7.04	15.90
44	11.8	6.1	15.9				
45	9.6	5.9	15.9	10.00	44.82	5.94	15.90
Profile 2							
27	10.0	9.7	15.9	10.00	27.00	9.66	15.90
28	13.4	11.0	15.9	13.20	27.94	10.89	15.90
29	15.9	12.0	15.9	15.90	29.00	11.97	15.90
30	16.6	12.7	15.9	16.60	30.00	12.65	15.90
31	15.0	11.7	15.9	15.00	31.00	11.70	15.90
32	13.2	11.0	15.9	13.20	32.00	10.95	15.90
33	9.7	9.7	15.9	10.00	32.91	9.78	15.90
42	13.9	8.3	15.9	13.90	42.00	8.30	15.90
43	13.5	7.4	15.9	13.20	43.18	7.29	15.90
44	11.8	6.6	15.9				
45	9.6		15.9	10.00	44.82		15.90
Profile 1							
27	10.0	8.8	15.9	10.00	27.00	8.80	15.90
28	13.4	9.1	15.9	13.40	28.00	9.10	15.90
29	15.9	9.3	15.9	15.90	29.00	9.30	15.90
30	16.6	8.7	15.9	16.60	30.00	8.70	15.90
31	15.0	9.1	15.9	15.00	31.00	9.10	15.90
32	13.2	9.0	15.9	13.40	31.89	9.01	15.90
33	9.7	8.5	15.9	10.00	32.91	8.54	15.90
42	13.9	8.9	15.9	13.90	42.00	8.90	15.90
43	13.5	8.2	15.9	13.40	43.06	8.15	15.90
44	11.8	7.3	15.9				
45	9.6	9.4	15.9	10.00	44.82	9.02	15.90

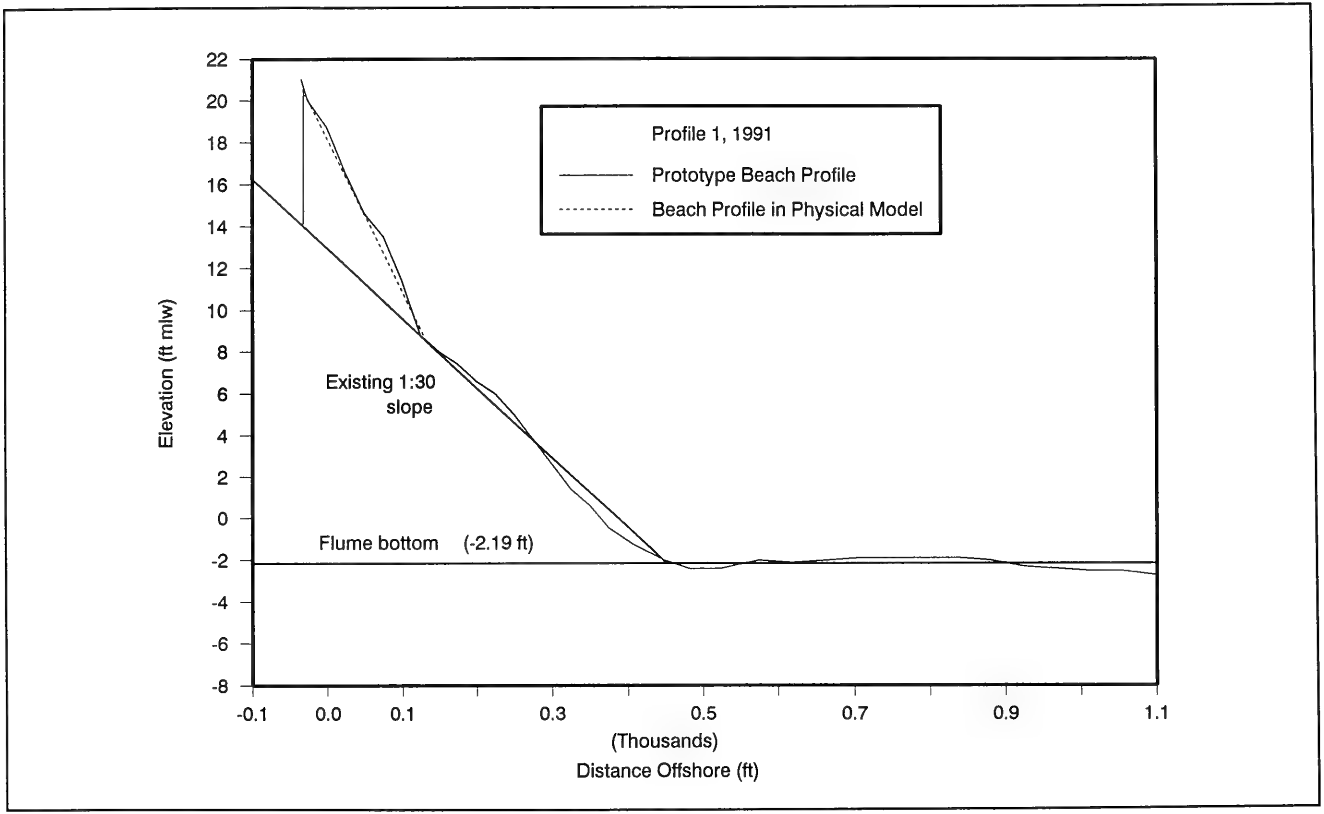

Figure 8. Profile 1 as surveyed in 1991 and as reproduced in physical model study

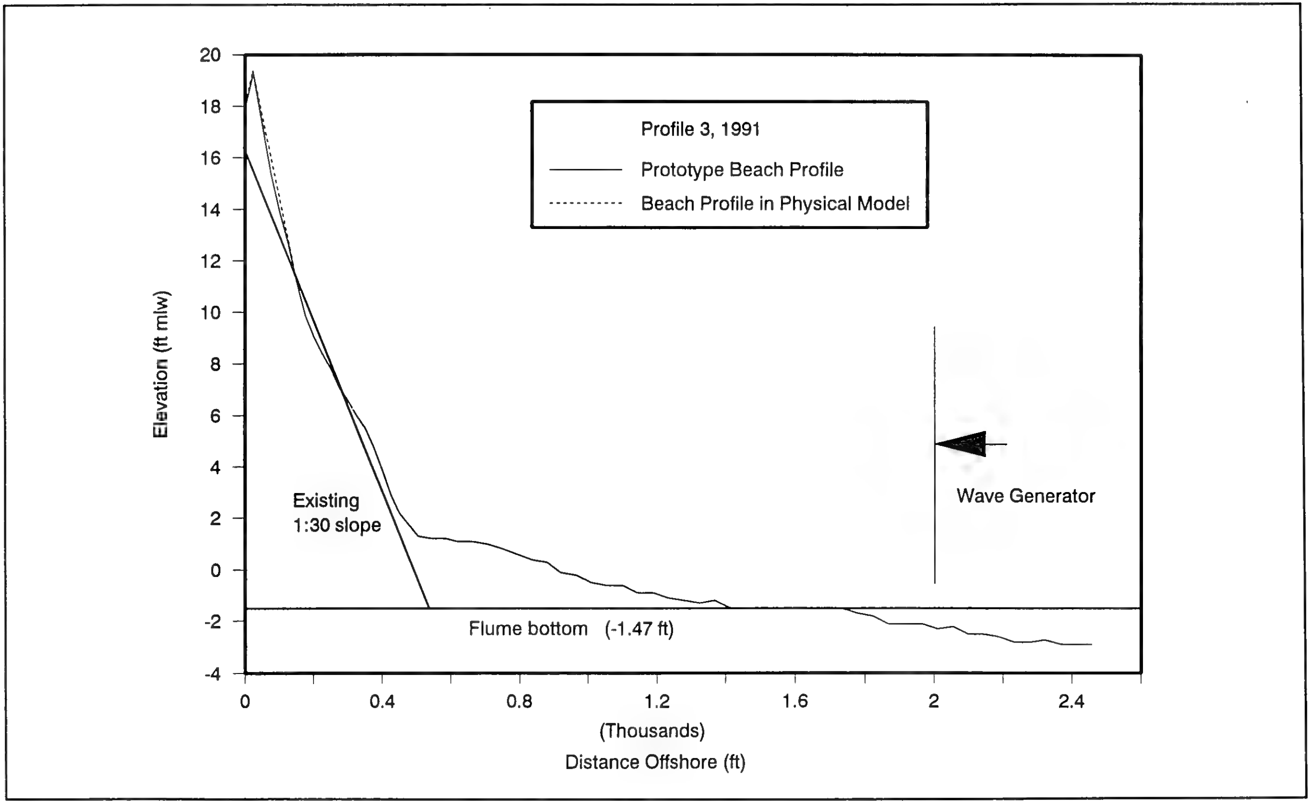

Figure 9. Profile 3 as surveyed in 1991 and as reproduced in physical model study

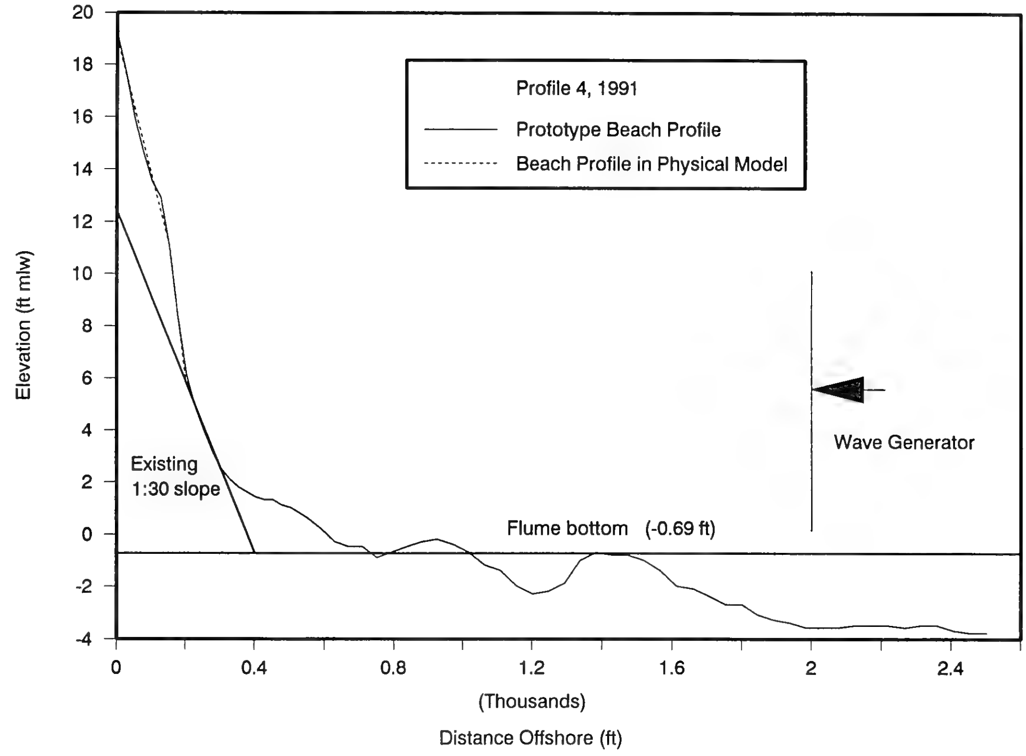

Figure 10. Profile 4 as surveyed in 1991 and as reproduced in physical model study

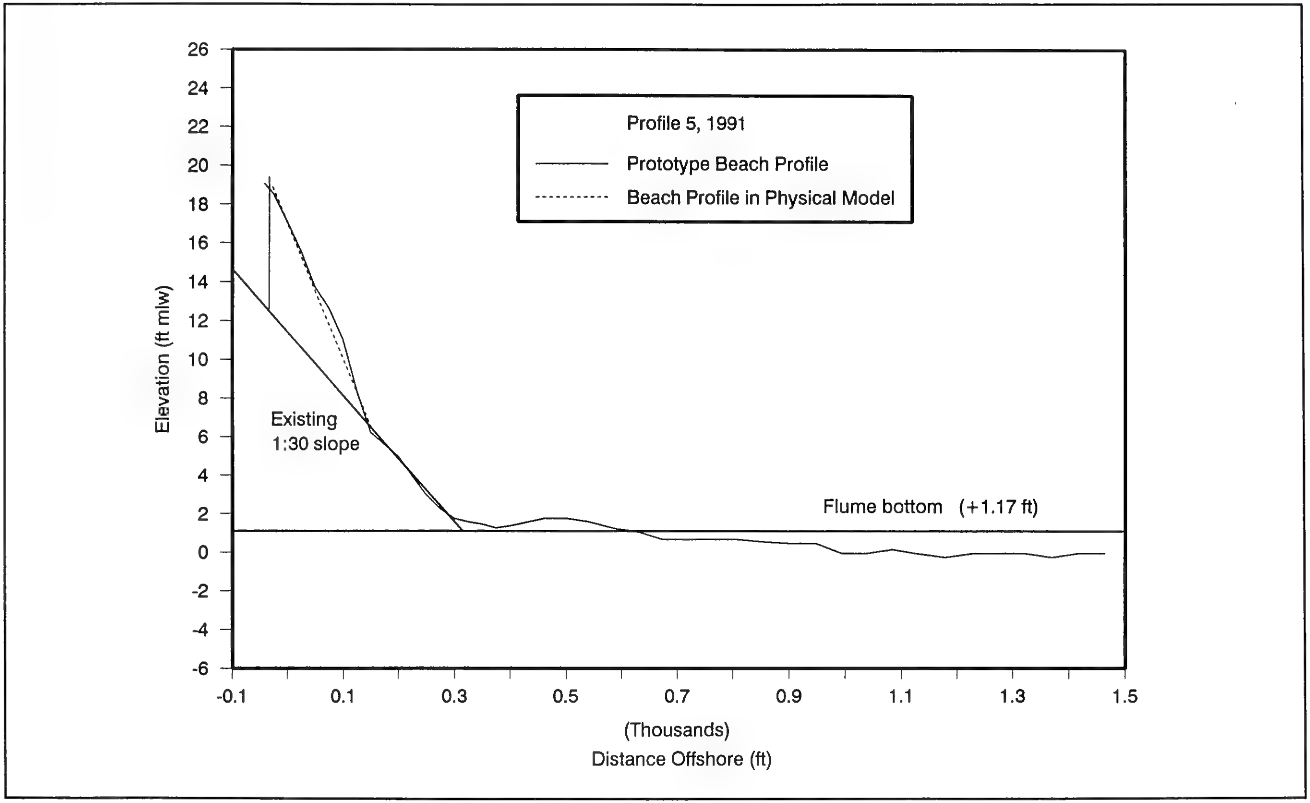

Figure 11. Profile 5 as surveyed in 1991 and as reproduced in physical model study

Table 5
Representative Seawall Crest Elevations for Overtopping Study of 1991 Beach Profiles

Profile No.	Seawall Elevation ft, mlw
1a	19.8
1b	20.7
2	21.3
3	20.6
4	20.3
5	20.4

an elevation of +19.4 ft, then an extension was added to continue the slope to an elevation of +20.3 ft, again providing a freeboard of 0.4 ft.

The wave generator in the 18-in. flume was unable to reproduce wave conditions at Profile 2 at a 1:30 scale. Rather than change to a smaller scale, Profile 2 was reproduced at a 1:30 scale in the 3-ft flume. Similar to the 18-in. flume, the existing 1:20 slope in the 3-ft flume was matched to a portion of the surveyed profile, and the steeper profile shoreward of the 1:20 slope was constructed of sheet metal. Surveyed and idealized profiles for Profile 2 are illustrated in Figure 12.

Results

Overtopping rate per linear foot of prototype seawall for each profile and each hour of the storm that had measurable overtopping are shown in Table 6. Physical model tests were not conducted on Profile 5 at hr 31, or Profiles 3 and 4 at hr 42. Volumes listed in Table 6 for these tests were obtained by multiple regression analysis using the other results listed in Table 6. Regression analysis is discussed below.

Storm conditions for the SPN were considerably worse than during the 1978 storm, with greater water depths and wave heights and longer wave periods. Overtopping rates, however, were considerably less, attesting to the incidental effectiveness of the 1991 beach fill. Overtopping rates measured in the wave flume for Profile 3 were surprisingly low, but incident wave heights for Profile 3 were lower than for the other profiles. NED confirmed that in the prototype, overtopping rates at Profile 3 appeared lower than at the other profiles during the October 30, 1991, storm, and the general trends observed in the wave flume agreed with observations of the prototype. The model did not test erosion of the 1991 beachfill during the SPN storm. Therefore, higher overtopping rates could be experienced as the beach erodes during the storm. Tests in Task C+ below show results if the beach should erode to 1978 contours.

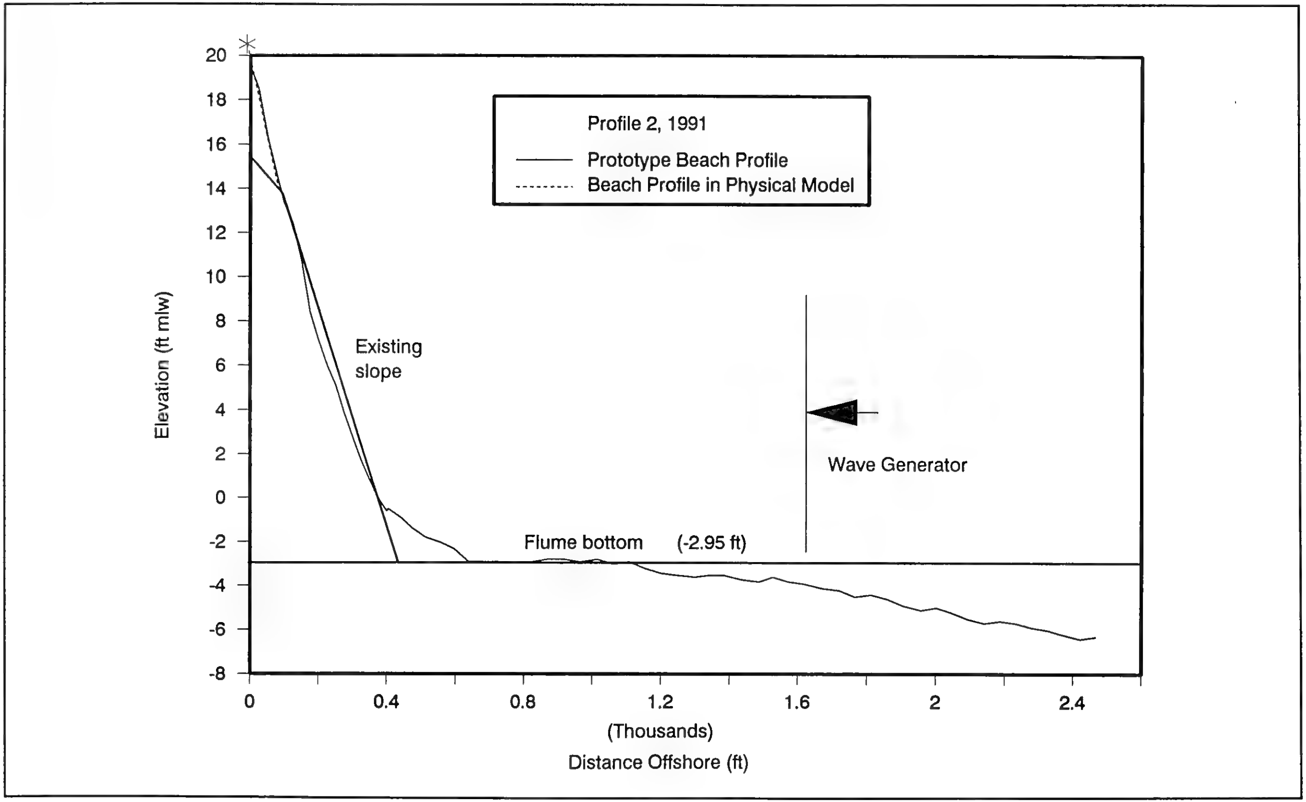

Figure 12. Profile 2 as surveyed in 1991 and as reproduced in physical model study

Table 6
Overtopping Rates and Volumes for SPN with 1991 Beach Profiles

Hour	Interp Hour	Begin Hour	End Hour	Total Seconds sec	Overtopping Rate, cfs/ft	Overtopping Volume, cf/ft
Profile 5						
29	29.00	28.50	29.50	3600	0.1947	701
30	30.00	29.50	30.50	3600	0.3928	1414
31[1]	31.00	30.50	31.44	3384	0.1729	585
Profile 4						
28	28.00	27.50	28.50	3600	0.0073	26
29	29.00	28.50	29.50	3600	0.3301	1189
30	30.00	29.50	30.50	3600	0.4070	1465
31	31.00	30.50	31.44	3400	0.1986	675
32	31.89	31.44	32.40	3446	0.0075	26
42[1]	42.00	41.50	42.53	3706	0.0553	205
43	43.06	42.50	43.50	3600	0.0078	28
Profile 3						
29	29.00	28.50	29.50	3600	0.0168	60
30	30.00	29.50	30.50	3600	0.0215	78
42[1]	42.00	41.50	42.53	3706	0.0000	0
43	43.06	42.53	43.50	3492	0.0022	8
Profile 2						
28	27.94	27.47	28.47	3600	0.0079	28
29	29.00	28.47	29.50	3706	0.2445	906
30	30.00	29.50	30.50	3600	0.4156	1496
31	31.00	30.50	31.50	3600	0.0955	344
32	32.00	31.50	32.46	3446	0.0109	38
42	42.00	41.50	42.59	3918	0.0150	59
43	43.18	42.59	43.50	3282	0.0043	14

(Continued)

[1] Determined by regression analysis.

Table 6 (Concluded)						
Hour	Interp Hour	Begin Hour	End Hour	Total Seconds sec	Overtopping Rate, cfs/ft	Overtopping Volume, cf/ft
Profile 1a						
28	28.00	27.50	28.50	3600	0.0336	121
29	29.00	28.50	29.50	3600	0.4311	1552
30	30.00	29.50	30.50	3600	0.8304	2989
31	31.00	30.50	31.44	3400	0.2337	794
32	31.89	31.44	32.40	3446	0.0571	197
42	42.00	41.50	42.53	3706	0.0904	335
43	43.06	42.53	43.50	3494	0.0364	127
Profile 1b						
28	28.00	27.50	28.50	3600	0.0215	78
29	29.00	28.50	29.50	3600	0.3105	1118
30	30.00	29.50	30.50	3600	0.5093	1834
31	31.00	30.50	31.44	3400	0.1329	452
32	31.89	31.44	32.40	3446	0.0258	89
42	42.00	41.50	42.53	3706	0.0396	147
43	43.06	42.53	43.50	3494	0.0155	54

Task C

Purpose

The purpose of Task C was to reproduce a selected set of conditions from a database of synthetic storm events (see Smith et al., in preparation). Data from Task C were used to develop a broken-wave overtopping module for use with numerical models of Revere Beach.

Selection of test conditions

CERC's Research Division selected storm conditions that were expected to produce overtopping from broken-wave runup. All tests were conducted on the model of the 1978 survey of Profile 2 in the 18-in. flume.

Test conditions selected by the Research Division for testing are listed in Table 7 as Tests 1 through 30. The selected tests were separated into groups with similar water depths to allow multiple tests to be conducted without changing water level in the wave flume. Table 7 also lists the actual test conditions used. The wave generator in the 18-in. flume was unable to produce the wave conditions for Tests 1 and 6; therefore, these tests were eliminated from the test series. Tests 25 and 26 were identical after adjusting the water level; therefore, Test 26 was deleted. The remaining tests were completed.

It was desired to perform a multiple regression analysis on the results of the tests to obtain a relationship among overtopping rate, wave height, wave period, and still-water level. Eight additional tests therefore were conducted to provide a better range of test conditions on which to base the analysis. The additional test conditions are shown in Table 7 as Tests 31 through 38.

At the conclusion of Task C, the Research Division asked that storm conditions selected from the SPN be tested with the 1978 profile. These additional tests were analyzed separately from Task C, and are therefore referred to in this report as Task C+. Six conditions representing peak hours of the storm were selected for testing. The wave generator in the 18-in. flume was unable to produce the wave heights at these conditions; therefore, tests were conducted at the highest obtainable H_{mo} for the given swl and T_p. Conditions tested and results are given in Table 7 as Tests 39 through 44.

Results

Results of the test series are given in Table 7.

Regression Analysis

Purpose

Regression analysis was performed on results of the physical model tests to determine relationships among overtopping rates, swl, and wave conditions. Regression analysis was conducted on results of Task B for the bore runup overtopping module, Task C (without C+, Tests 39 through 44) for the broken wave overtopping module, combined results of tasks A, C, and C+ for a "worst case" analysis, and on the entire set of tests. The Statistical Analysis System (SAS), version 6.04, was used for the analysis.

Table 7
Test Conditions and Overtopping Rates for Task C

Test No.	SWL, ft, mlw	SWL Tested ft, mlw	Wave Height ft	Wave Period sec	Overtopping Rate, cfs/ft
1	14.9	14.9	10.5	9.0	
2	14.6	14.6	7.1	11.7	0.6431
3	14.6	14.6	8.1	9.0	0.5498
4	14.6	14.6	10.6	11.3	0.7475
5	14.2	14.1	8.5	12.7	0.5472
6	14.2	14.1	11.7	13.4	
7	14.1	14.1	6.1	8.7	0.2550
8	14.1	14.1	8.1	9.0	0.4331
9	14.0	14.1	9.0	10.7	0.6230
10	13.5	13.4	8.5	12.7	0.3711
11	13.3	13.4	7.6	12.0	0.3002
12	13.3	13.4	7.8	9.0	0.2530
13	13.3	13.4	7.9	12.3	0.2970
14	13.2	13.1	7.7	11.3	0.2397
15	13.1	13.1	7.8	12.0	0.2589
16	13.0	13.1	4.7	11.3	0.1380
17	13.0	13.1	8.1	13.0	0.2679
18	12.9	13.1	6.6	9.0	0.2001
19	12.6	12.6	7.6	11.7	0.1658
20	12.1	12.0	7.5	14.1	0.1109
21	12.0	12.0	6.5	8.3	0.0889
22	11.9	12.0	8.1	12.0	0.1098
23	11.6	11.6	3.1	11.0	0.0278
24	11.6	11.6	7.1	9.0	0.0691
25	11.6	11.6	7.3	12.0	0.0812
26	11.5	11.6	7.3	12.0	
27	10.9	10.7	7.4	13.0	0.0334
28	10.7	10.7	5.5	9.0	0.0274
29	10.5	10.7	6.7	8.6	0.0314
30	9.5	9.5	6.5	14.1	0.0052

(Continued)

Table 7 (Concluded)					
Test No.	SWL, ft, mlw	SWL Tested ft, mlw	Wave Height ft	Wave Period sec	Overtopping Rate, cfs/ft
31		14.2	6.00	12.70	0.4309
32		14.2	6.00	9.00	0.2795
33		14.2	6.00	10.70	0.3192
34		13.2	5.10	13.00	0.2018
35		13.2	5.40	9.00	0.1396
36		13.2	6.21	11.30	0.2061
37		12.1	6.00	14.10	0.1049
38		12.1	7.80	8.30	0.0963
39		15.9	8.38	15.90	0.8489
40		16.6	8.86	15.90	1.1980
41		15.0	9.36	15.90	1.0337
42		13.2	8.76	15.90	0.4049
43		10.0	7.82	15.90	0.3915
44		13.2	7.29	15.90	0.0181

Method

Dimensionless parameters were selected that were suitable for the numerical models for which the regression models were destined. Overtopping rate was presented as

$$Q \text{ [=] cfs/ft} = L^2T^{-1}$$

where

Q = overtopping rate

[=] = appropriate dimensional units

Dimensional parameters affecting overtopping rate include the following:

$$f \text{ [=] ft} = L$$

$$b \text{ [=] ft} = L$$

$$d \text{ [=] ft} = L$$

$$H \text{ [=] ft} = L$$

T [=] sec = T

g [=] ft/sec^2 = LT^{-2}

where

f = structure freeboard defined as height of the seawall crest above the swl

b = beach freeboard defined as height of the beach at the base of the seawall above the swl

d = water depth at the flat bottom of the wave flume

H = wave height defined as the monochromatic wave height at a distance of 2,000 ft offshore (approximate location of wave generator in model flume tests and the wave height on which the physical model tests were based)

T = wave period associated with the monochromatic wave height H

g = gravitational acceleration

Dimensionless parameters that may also affect overtopping rates include:

$\cot\theta$

$d/d2000$

where

$\cot\theta$ = cotangent of the beach slope defined as cotangent of the slope from the base of the seawall to the swl

$d2000$ = depth at a distance of 2,000 ft offshore

Because the model profiles did not extend to the wave generator, there was a difference in depth between the wave generator in the flume (adjusted for scale) and the actual depth offshore of Revere Beach. The ratio $d/d2000$ is the ratio of the depth in the flume (adjusted for scale) to the depth where the wave heights were determined from the numerical model. Figures 8 through 12 show where the flume bottoms were fitted to the beach profiles and illustrate the differences between depths in the flumes and depths on the surveys at the location of the wave generator. Because input wave information (wave height and period) was obtained from SBEACH at the approximate location of the wave generator (approximately 2,000 ft offshore from the seawall), it was thought that the difference in depths, $d/d2000$, could play a role in defining the overtopping rates. Depths at 2,000 ft offshore varied somewhat throughout the storm due to sediment movement; therefore, the depth determined by SBEACH for each hour of the storm was used for analysis. The profile in the flume, of course, remained constant.

Task B

All wave flume tests conducted for the SPN used a wave period of 15.9 sec. Because this value was a constant for all tests, it was not used in the analysis. Gravitational acceleration was therefore the only term available by which to nondimensionalize overtopping rate in time. All other parameters required only a length scale, and either f or H were reasonable candidates for the repeating variable. After trying both variables, it was found that results were somewhat improved by using f. After many variations and combinations of terms were tried, the dimensionless variables that provided the best fit to the data were arranged as follows:

$$Q' = Q/(g*f^3)^{1/2}$$

$$PI1 = b/f$$

$$PI2 = H/f$$

$$PI3 = d/d2000$$

$$PI4 = \cot$$

$$PI5 = d/f$$

Data collected in the physical model tests were converted to prototype scale for the regression analysis. Input data are shown in Table 8. Note that the last three lines in Table 8 give the input data for the three points in Table 6 determined by regression analysis.

Examination of the residuals from one of the regression models that was tried indicated that higher-order terms were required (a residual is the difference between Q' predicted by the regression model and measured Q'). Second-order terms (squares of the PI variables) and higher were therefore added to the analysis.

Regression analysis was conducted on the dimensionless variable Q'. Any negative overtopping rates predicted were set to zero, and results were converted to predicted dimensional overtopping rates. Model selection was then based on the sum of squares of differences between observed and predicted overtopping rates.

SAS assumes a null hypothesis that the coefficient of a term in the model is zero, then computes the probability that the null hypothesis is true. Only terms in the model with a low probability of having zero

Table 8
Wave Conditions and Seawall Elevations for 1991 Profiles of Revere Beach[1]

Profile No.	Storm Hour	SWL ft, mlw	Depth in Flume, ft	Wave Height ft	Wave Period sec	Seawall Crest Elev. sec	Base of Seawall ft, mlw	Over-topping cfs/ft	Cotan Beach Slope	Elev at 2,000 ft Offshore ft, mlw
5	28	13.4	12.23	9.10	15.9	20.4	19.1	0.0000	14.5	-1.40
5	29	15.9	14.73	8.90	15.9	20.4	19.1	0.1947	14.5	-1.40
5	30	16.6	15.43	8.70	15.9	20.4	19.1	0.3928	14.5	-1.40
4	27	10.0	10.69	7.40	15.9	20.3	19.1	0.0000	19.5	-3.57
4	28	13.4	14.09	7.90	15.9	20.3	19.1	0.0073	19.5	-3.57
4	29	15.9	16.59	10.40	15.9	20.3	19.1	0.3301	19.5	-3.57
4	30	16.6	17.29	7.10	15.9	20.3	19.1	0.4070	19.5	-3.57
4	31	15.0	15.69	9.60	15.9	20.3	19.1	0.1986	19.5	-3.57
4	32	13.4	14.09	7.91	15.9	20.3	19.1	0.0075	19.5	-3.57
4	43	13.4	14.09	6.96	15.9	20.3	19.1	0.0078	19.5	-3.56
3	29	15.9	17.37	4.50	15.9	20.5	18.2	0.0168	15.5	-2.25
3	30	16.6	18.07	4.40	15.9	20.5	18.2	0.0215	15.5	-2.25
3	32	13.4	14.87	4.40	15.9	20.5	18.2	0.0000	15.5	-2.25
3	43	13.4	14.87	7.04	15.9	20.5	18.2	0.0022	15.5	-2.23
2	28	13.2	16.15	10.89	15.9	21.3	20.5	0.0079	16.0	-5.03
2	29	15.9	18.85	11.97	15.9	21.3	20.5	0.2445	16.0	-5.02
2	30	16.6	19.55	12.65	15.9	21.3	20.5	0.4156	16.0	-5.02
2	31	15.0	17.95	11.70	15.9	21.3	20.5	0.0955	16.0	-5.02
2	32	13.2	16.15	10.95	15.9	21.3	20.5	0.0109	16.0	-5.02
2	33	10.0	12.95	9.78	15.9	21.3	20.5	0.0000	16.0	-5.02
2	42	13.9	16.85	8.30	15.9	21.3	20.5	0.0150	16.0	-5.04
2	43	13.2	16.15	7.29	15.9	21.3	20.5	0.0043	16.0	-5.04
1	28	13.4	15.59	9.10	15.9	19.8	19.4	0.0336	14.0	-5.42
1	29	15.9	18.09	9.30	15.9	19.8	19.4	0.4311	14.0	-5.41
1	30	16.6	18.79	8.70	15.9	19.8	19.4	0.8304	14.0	-5.41
1	31	15.0	17.19	9.10	15.9	19.8	19.4	0.2337	14.0	-5.41
1	32	13.4	15.59	9.01	15.9	19.8	19.4	0.0571	14.0	-5.41
1	42	13.9	16.09	8.90	15.9	19.8	19.4	0.0904	14.0	-5.45
1	43	13.4	15.59	8.15	15.9	19.8	19.4	0.0364	14.0	-5.45
1	45	10.0	12.19	9.02	15.9	19.8	19.4	0.0000	14.0	-5.45
1	28	13.4	15.59	9.10	15.9	20.7	20.3	0.0215	14.0	-5.42
1	29	15.9	18.09	9.30	15.9	20.7	20.3	0.3105	14.0	-5.41

(Continued)

[1] All measurements are prototype scale.

Table 8 (Concluded)										
Profile No.	Storm Hour	SWL ft, mlw	Depth in Flume, ft	Wave Height ft	Wave Period sec	Seawall Crest Elev. sec	Base of Seawall ft, mlw	Over-topping cfs/ft	Cotan Beach Slope	Elev at 2,000 ft Offshore ft, mlw
1	30	16.6	18.79	8.70	15.9	20.7	20.3	0.5093	14.0	-5.41
1	31	15.0	17.19	9.10	15.9	20.7	20.3	0.1329	14.0	-5.41
1	32	13.4	15.59	9.01	15.9	20.7	20.3	0.0258	14.0	-5.41
1	42	13.9	16.09	8.90	15.9	20.7	20.3	0.0396	14.0	-5.45
1	43	13.4	15.59	8.15	15.9	20.7	20.3	0.0155	14.0	-5.45
1	45	10.0	12.19	9.02	15.9	20.7	20.3	0.0000	14.0	-5.45
5	31	15.0	13.83	8.30	15.9	20.4	19.1		14.5	-1.40
4	42	13.9	14.59	9.90	15.9	20.3	19.1		19.5	-3.57
3	42	13.9	15.37	7.30	15.9	20.5	18.2		15.5	-2.23

coefficients (typically 10 percent for this study) were retained in the selected models.

The model that best fit the data in Task B was:

$$Q' = -0.0190100 + 0.113943*PI1 - 0.074790*PI1^2$$
$$+ 0.114503*PI3^2 - 0.072397*PI3^4$$
$$- 0.007017*PI4 + 0.000199*PI4^2$$
$$- 0.006809*PI5 + 0.001601*PI5^2$$

While this equation was somewhat tedious, it fit the data with a correlation coefficient of 0.991 ($R^2 = 0.983$), and the sum of squares of differences between the overtopping rates (dimensional) and measured overtopping was only 0.074. There were 38 data points in the analysis; therefore, the average difference between calculated and measured overtopping was ± 0.044 cfs/ft.

It should be emphasized that regression models presented in this report are site-specific and are only valid at Revere Beach and within the range of conditions tested. The range of variables used, both dimensional and nondimensional, is given in Table 9.

It seemed unreasonable to delete wave height (*PI*2) from the model, especially when a correlation analysis revealed that Q' was more highly correlated with dimensionless wave height than any other single variable. However, there was a very high correlation between dimensionless wave height and dimensionless water depth (*PI*2 and *PI*5, 76-percent correlation),

Table 9
Minimum and Maximum Values for Parameters Used in Regression Analysis of 1991 Overtopping Rates

Parameter	Min	Max
SWL, ft mlw	10.00	16.60
Wave height, ft	4.40	12.65
Wave period, sec	15.90	15.90
Seawall freeboard, ft	3.20	11.30
Beach freeboard, ft	1.60	10.50
Cotan beach slope	14.00	19.50
Overtopping rate, cfs/ft	0.0000	0.8304
PI1	0.4102	0.9626
PI2	0.6197	2.6915
PI3	0.7878	0.9586
PI4	14.0000	19.5000
PI5	1.0379	4.6730
Q'	0.0000	0.0101

which was expected for depth-limited breaking waves, and effects of wave height were therefore reflected in *PI5*.

A much simpler model provided a reasonable fit to the data and used only *PI1* and *PI5* (beach elevation and water depth). Initial analysis of the data revealed that overtopping rates for hr 30, Profile 1, at both seawall crest elevations were exerting a very high influence on the simplified regression model. Because these overtopping rates were extreme and will not be found in other storms for which the regression model will be used, these two values were excluded from the analysis.

The model was:

$$Q' = -0.036533 + 0.099865*PI1 - 0.062324*PI^2$$

$$-0.003554*PI5 + 0.001114*PI5^2$$

This very simple model fit the data with a correlation coefficient of 0.969 (R^2 = 0.939), sum of squares of the dimensional errors was 0.0796, and average difference between calculated and measured overtopping rates was ± 0.047 cfs/ft. The exclusion of beach slope in this simplified model was probably due to the small range of the variable (14.0 to 19.5) and the relatively short distance that the slope was used in the wave flume.

This simplified model was used by CERC's Research Division for the bore runup overtopping module.

Task C

Data from Task C (excluding C+, Tests 39 through 44) were analyzed to determine a regression model for a broken wave overtopping module. Input conditions for the regression analysis (in prototype scale) are given in Table 10.

Overtopping rate was nondimensionalized in the same manner in Task B, that is, as $Q' = Q/(g*f^3)^{1/2}$. Other variables that were determined to be significant in the regression analysis were:

$PI1 = \mathrm{swl}/f$

$PI2 = H/f$

$PI3 = L_o/f$

where L_o is deepwater wavelength defined as

$L_o = (g/(2\pi)*T^2$

The model that gave the best results was weighted by wave height and is given as

$$Q' = 0.004162 - 0.007285*PI1 + 0.003252*PI1^2 + 0.001559*PI2^2 - 0.000025997*PI3 + 0.000000217*PI3^2$$

As in Task B, this model was selected based on the sum of squares of residuals of the dimensional overtopping rates. Correlation coefficient for the nondimensional model was 0.9865 ($R^2 = 0.9732$). Sum of squares of residuals for the dimensional overtopping was 0.0511 for 35 test runs, yielding an average error of ± 0.038 cfs/ft.

It should again be emphasized that the regression analysis should not be used beyond the limits of the data set or for any other sites. Table 11 lists the ranges of variables used in the analysis.

1978 Profile

All tests conducted using the 1978 survey of Profile 2 were combined in a single data set for analysis. The data included Tasks A and C plus the

Table 10
Input Data for Revere Beach Overtopping Rates Regression Analysis, Task C, Profile No. 2, Survey Year 1978

SWL, ft mlw	Wave Height, ft	Wave Period, sec	Overtopping Rate cfs/ft
14.90	10.50	9.00	
14.60	7.10	11.70	0.6431
14.60	8.10	9.00	0.5498
14.60	10.60	11.30	0.7475
14.10	8.50	12.70	0.5472
14.10	11.70	13.40	
14.10	6.10	8.70	0.2550
14.10	8.10	9.00	0.4331
14.10	9.00	10.70	0.6230
13.40	8.50	12.70	0.3711
13.40	7.60	12.00	0.3002
13.40	7.80	9.00	0.2530
13.40	7.90	12.30	0.2970
13.10	7.70	11.30	0.2397
13.10	7.80	12.00	0.2589
13.10	4.70	11.30	0.1380
13.10	8.10	13.00	0.2679
13.10	6.60	9.00	0.2001
12.60	7.60	11.70	0.1658
12.00	7.50	14.10	0.1109
12.00	6.50	8.30	0.0889
12.00	8.10	12.00	0.1098
11.60	3.10	11.00	0.0278
11.60	7.10	9.00	0.0691
11.60	7.30	12.00	0.0812

(Continued)

Note: Seawall Crest Elev. = 21.0 ft mlw; Elev. Base of Seawall = 9.2 ft mlw;
Cotan Beach Slope = 10.7; Elev. at Flume Bottom = -3.00 ft mlw.

Table 10 (Concluded)

SWL, ft mlw	Wave Height, ft	Wave Period, sec	Overtopping Rate cfs/ft
10.70	7.40	13.00	0.0334
10.70	5.50	9.00	0.0274
10.70	6.70	8.60	0.0314
9.50	6.50	14.10	0.0052
14.10	6.00	12.70	0.4309
14.10	6.00	9.00	0.2795
14.10	6.00	10.70	0.3192
13.10	5.10	13.00	0.2018
13.10	5.40	9.00	0.1396
13.10	6.21	11.30	0.2061
12.00	6.00	14.10	0.1049
12.00	7.80	8.30	0.0963

Table 11
Minimum and Maximum Values for Parameters in the Regression Analysis for Task C

Parameter	Min	Max
SWL, ft mlw	9.5	14.9
Wave height, ft	3.1	11.7
Wave period, sec	8.3	14.1
Seawall freeboard, ft	6.1	11.5
Beach freeboard, ft	-5.7	-0.3
Overtopping rate, cfs/ft	0.0052	0.7475
PI1	0.826	2.443
PI2	0.330	1.721
PI3	36.764	133.239

additional tests conducted after Task C listed as C+. The data set for this effort is given in Table 12.

Dimensionless variables that yielded the best results were similar to those used in the regression analysis of Task B, above. Dimensionless overtopping was defined in the same way, and the repeating variable was again seawall freeboard (distance between seawall crest elevation and swl). Dimensionless beach elevation used in Task B was replaced with the dimensionless difference between seawall crest elevation and beach elevation, and *PI*4 was deleted because beach slope for the 1978 profile was constant. Wave period was a factor, and was characterized by deep-water wavelength. Depth in the flume and swl differed by a constant; therefore, they could not both be used and swl was selected for *PI*5. The dimensionless variables are listed below.

$$Q' = Q/(g^*f^3)^{1/2}$$

$$PI1 = (f-b)/f$$

$$PI2 = H/f$$

$$PI5 = \mathrm{swl}/f$$

$$PI6 = L_o/f$$

The dimensionless variable *PI*3 (*d*/*d*2000) used in Task B was not included because data for *d*2000 were not available for conditions in Task C.

In conducting the analysis, one point was found to lie outside the general trend. In the set of six tests conducted as Task C+, the measured overtopping from the test with an swl of 15.0 ft was substantially greater than predicted. This data point yielded an unacceptable influence on the results, and was therefore deleted from the analysis. For the remaining data, the selected regression model is given below.

$$Q' = -0.000338 + 0.002530{*}PI1^4 - 0.004788{*}PI5^2$$
$$+ 0.001912{*}PI2^3 - 0.000322{*}PI2^6$$
$$+ 0.000000212{*}PI6^2 - 6.92016{*}10^{-12}{*}PI6^4$$

This model was selected by weighting the analysis by wave period, thereby increasing the significance of longer period waves.

This model had a correlation coefficient of 0.992 ($R^2 = 0.984$), and the correlation coefficient of dimensional overtopping (measured to predicted) was 0.970. Sum squares of the residuals of dimensional overtopping was 0.3106 for the 60 tests; therefore, the average error was 0.072 cfs/ft.

Table 12
Input Data for Revere Beach Overtopping Rates Regression Analysis, Profile No. 2, Survey Year 1978

Task	SWL, ft mlw	Wave Height, ft	Wave Period, sec	Overtopping Rate, cfs/ft
A	10.82	6.14	8.10	0.0066
A	12.96	6.99	8.60	0.0643
A	12.96	10.05	9.70	0.1004
A	10.82	5.99	11.00	0.0080
A	10.82	6.00	12.30	0.0077
A	12.96	6.76	12.60	0.0843
A	15.40	7.65	13.00	1.3553
A	12.96	6.79	13.00	0.0959
A	10.82	6.01	12.90	0.0063
A	11.00	6.35	8.25	0.0097
A	13.15	7.18	8.59	0.0994
A	14.30	10.80	9.00	0.5200
A	13.15	10.18	9.62	0.1659
A	11.00	6.39	10.45	0.0103
A	11.00	6.34	12.44	0.0100
A	13.15	7.09	12.74	0.1674
A	14.75	7.70	13.00	0.8141
A	14.75	7.70	13.00	0.8141
A	13.15	7.11	13.00	0.1359
A	11.00	6.30	13.00	0.0116
C	14.90	10.50	9.00	
C	14.60	7.10	11.70	0.6431
C	14.60	8.10	9.00	0.5498
C	14.60	10.60	11.30	0.7475
C	14.10	8.50	12.70	0.5472
C	14.10	11.70	13.40	
C	14.10	6.10	8.70	0.2550
C	14.10	8.10	9.00	0.4331
C	14.10	9.00	10.70	0.6230
C	13.40	8.50	12.70	0.3711
C	13.40	7.60	12.00	0.3002
C	13.40	7.80	9.00	0.2530

(Continued)

Note: Seawall Crest Elev. = 21.0 ft mlw; Elev. Base of Seawall = 9.2 ft mlw; Cotan Beach Slope = 10.7; Elev. at Flume Bottom = -3.00 ft mlw.

Table 12 (Concluded)				
Task	SWL, ft mlw	Wave Height, ft	Wave Period, sec	Overtopping Rate, cfs/ft
C	13.40	7.90	12.30	0.2970
C	13.10	7.70	11.30	0.2397
C	13.10	7.80	12.00	0.2589
C	13.10	4.70	11.30	0.1380
C	13.10	8.10	13.00	0.2679
C	13.10	6.60	9.00	0.2001
C	12.60	7.60	11.70	0.1658
C	12.00	7.50	14.10	0.1109
C	12.00	6.50	8.30	0.0889
C	12.00	8.10	12.00	0.1098
C	11.60	3.10	11.00	0.0278
C	11.60	7.10	9.00	0.0691
C	11.60	7.30	12.00	0.0812
C	10.70	7.40	13.00	0.0334
C	10.70	5.50	9.00	0.0274
C	10.70	6.70	8.60	0.0314
C	9.50	6.50	14.10	0.0052
C	14.10	6.00	12.70	0.4309
C	14.10	6.00	9.00	0.2795
C	14.10	6.00	10.70	0.3192
C	13.10	5.10	13.00	0.2018
C	13.10	5.40	9.00	0.1396
C	13.10	6.21	11.30	0.2061
C	12.00	6.00	14.10	0.1049
C	12.00	7.80	8.30	0.0963
C+	15.90	8.38	15.90	0.8489
C+	16.60	8.86	15.90	1.1980
C+	15.00	9.36	15.90	1.0337
C+	13.20	8.76	15.90	0.4049
C+	10.00	7.82	15.90	0.3915
C+	13.20	7.29	15.90	0.0181

As with all regression models presented in this report, this model is only valid for the range of conditions tested and for the specific project site. The range of variables, both dimensional and nondimensional, used in this analysis is given in Table 13.

Table 13
Minimum and Maximum Values for Parameters in the Regression Analysis with 1978 Profile

Parameter	Min	Max
SWL, ft mlw	9.5	16.6
Wave height, ft	3.1	11.7
Wave period, sec	8.1	15.9
Seawall freeboard, ft	4.4	11.5
Beach freeboard, ft	-7.4	-0.3
Overtopping rate, cfs/ft	0.0052	1.3553
PI1	1.026	2.682
PI2	0.330	2.014
PI5	0.826	3.773
PI6	33.029	294.454
Q'	0.0000	0.0229

Combined regression analysis for Tasks A, B, C, and C+

All data collected in Tasks A, B, C, and C+ were combined into a single data set to develop a general regression model for the overtopping at Revere Beach. The combined data set includes all data listed in Tables 8 and 12, with the exception of the outlier mentioned above under 1978 profile. The same dimensionless parameters used in the analysis of the 1978 profile were used in the current analysis, but *PI*4 was added to include the beach slope. The variables are therefore defined as

$$Q' = Q/(g^*f^3)^{1/2}$$

$$PI1 = (f\text{-}b)/f$$

$$PI2 = H/f$$

$$PI4 = \cot\theta$$

$$PI5 = \mathrm{swl}/f$$

$PI6 = L_o/f$

The regression model that provided the best fit to the remaining data is given below. The model was weighted by deepwater wavelength.

$$
\begin{aligned}
Q' = 0.035883 &- 0.010479*PI1 + 0.005523*PI1^2 \\
&- 0.003424*PI2 + 0.001962*PI2^2 \\
&- 0.004667*PI4 + 0.000142*PI4^2 \\
&+ 0.000230*PI5^2 + 0.000000536*PI5^4 \\
&+ 0.000068128*PI6 - 0.000000290*PI6^2
\end{aligned}
$$

This model had a correlation coefficient of 0.976 ($R^2 = 0.952$). The sum of squares of differences between predicted and measured dimensional overtopping rates was 0.887, which, for 98 data points, yielded an average difference of ± 0.095 cfs/ft.

As with all regression models presented in this report, this model is only valid for the range of conditions tested and is site specific for Revere Beach. The range of variables, both dimensional and nondimensional, used in this analysis is given in Table 14.

Table 14
Minimum and Maximum Values for Parameters Used in Regression Analysis of Combined Data from Tasks A, B, and C

Parameter	Min	Max
SWL, ft mlw	9.5000	16.6000
Wave height, ft	3.1000	12.6500
Wave period, sec	8.1000	15.9000
Seawall freeboard, ft	3.2000	11.5000
Freeboard between seawall and beach, ft	0.4000	11.8000
Cotan beach slope	10.7000	19.5000
Overtopping rate, cfs/ft	0.0000	1.3553
PI1	0.0374	2.6818
PI2	0.3298	2.7188
PI4	10.7000	19.5000
PI5	0.8261	5.1875
PI6	33.0292	404.8744
Q'	0.0000	0.0256

4 Discussion of Research Tasks A, B, and C

A major uncertainty in the physical model tests was the wave spectrum being tested. Wave information furnished for the storm profile consisted of a monochromatic wave height and wave period, obtained by refracting and diffracting a representative wave (peak period and significant wave height) of a wave spectrum. This representative wave, after shoaling to the approximate distance offshore modeled by the wave generator in the wave flume, then was used as the peak period and zeroth moment wave height to reproduce a new irregular wave spectrum. This would be accurate if the entire spectrum shoaled to the same extent as the representative wave. In reality, each frequency in the incident spectrum will shoal differently, and an entirely new spectrum will exist after shoaling. Although we have the capability of dividing the incident spectrum into a number of bandwidths, transforming each bandwidth individually through numerical models REF/DIF and SBEACH, and then reassembling the transformed spectrum from the individual bandwidths, the procedure is time-consuming, not economically feasible, and other uncertainties in the prototype and physical model do not justify attempting such a level of precision. This uncertainty applied to Tasks A, B, C, and C+, and the net effect on overtopping rate caused by this approximation of the wave spectrum is unknown.

With the 1978 profiles (Tasks A, C, and C+), there was considerable freeboard between the beach and seawall crest. Waves striking the seawall were forced into a vertical sheet of water and spray, frequently exceeding the height of the seawall. Because the motion was nearly vertical, much of this water fell back on the seaward side of the seawall in the flume, but wind effects may cause more of the water to overtop the seawall in the prototype.

Wind effects on overtopping rates in Task B are expected to be minimal. Wind has two effects on seawall overtopping rates: modification of wave runup on the beach, and blowing spray over the seawall. Modification of the wave runup has been calculated to have only a minor effect on overtopping[1]. Due to the low freeboard between the 1991 beach profiles and seawall crest elevations, waves overtopping the seawall tended to flow over the wall in a bore rather than be deflected vertically as in Tasks A, C, and C+. Because the water movement was horizontal rather than vertical, wind effects are not expected to be significant.

Due to high reflection coefficients from the high seawall freeboard in models of the 1978 profile, wave energy reflected from the seawall remained in the wave flume and increased the total energy in the flume over time. Avoiding this effect would require that each test run be terminated before energy reflected from the structure could reach the wave generator and return to the structure. Each test then would be on the order of 2 min, after which the testing would be halted until the energy in the flume had dissipated. A series of short tests then would be used to ensure that the entire wave spectrum was represented. Again, this level of accuracy is probably not justified, and would be time-consuming and expensive. The probable effect of this increased energy level in the flume is an increase in overtopping rates for the 1978 profile tests.

Because of the low seawall freeboards and extended beach profiles compared to Tasks A, C, and C+, reflection coefficients for Task B were small and reflected wave energy was not a significant factor in the tests.

[1] Memorandum to Joan Pope entitled "Assessment of wind effects on wave overtopping of proposed Virginia Beach seawall," 1987, from Donald T. Resio, Offshore and Coastal Technologies, Inc., Vicksburg, MS.

5 Revere Dike Study

Park Dike, 1991 Profile

Model construction

Plans for the proposed park dike were received at CERC from NED, and are reproduced in part in Figure 13. The plans propose narrowing Revere Boulevard and adjacent sidewalks from the current 80 ft to 46 ft and building a mound of random fill covered with a 12-in. layer of topsoil. Design of the mound specified a crest elevation of +27.5 ft mlw (equivalent to 23.0 ft NGVD as seen in Figure 13, where NGVD is National Geodetic Vertical Datum), a toe wall along the seaward toe at elevation +21.5 ft mlw, a width of approximately 110 ft from toe wall to crest yielding a slope of approximately 1:18.3, and a slope of 1:2.5 on the landward side of the dike. Within the dike is a rubble core to provide protection in case the topsoil and random fill are eroded away during a storm.

In the model, a toe wall was constructed to an elevation of +21.5 ft mlw at a distance of 46 ft shoreward of the seawall as specified in the park dike plans. The dike was constructed with plywood extending from the crest of the toe wall to the crest of the dike, with the area shoreward of the dike crest sealed to retain any overtopping (Figure 14).

All tests were conducted by first filling the area between the seawall and toe of the park dike with water to allow maximum wave energy to reach the dike. Seawall elevations of +20.9 and +21.3 ft mlw are found in the reach represented by Profile 2; both seawall elevations were tested.

Three park dike crest elevations were tested. The highest crest elevation was the proposed elevation of +27.5 ft mlw, the lowest elevation was set at +24.0 ft for a seaward slope of about 0.014, and the third elevation was approximately midway at +25.6 ft mlw.

The model was tested with the 1991 post-storm beach profile which had been constructed during Task B, above, for Profile 2. The beach profile had a beach elevation at the seawall of +20.5 ft mlw. Additional tests

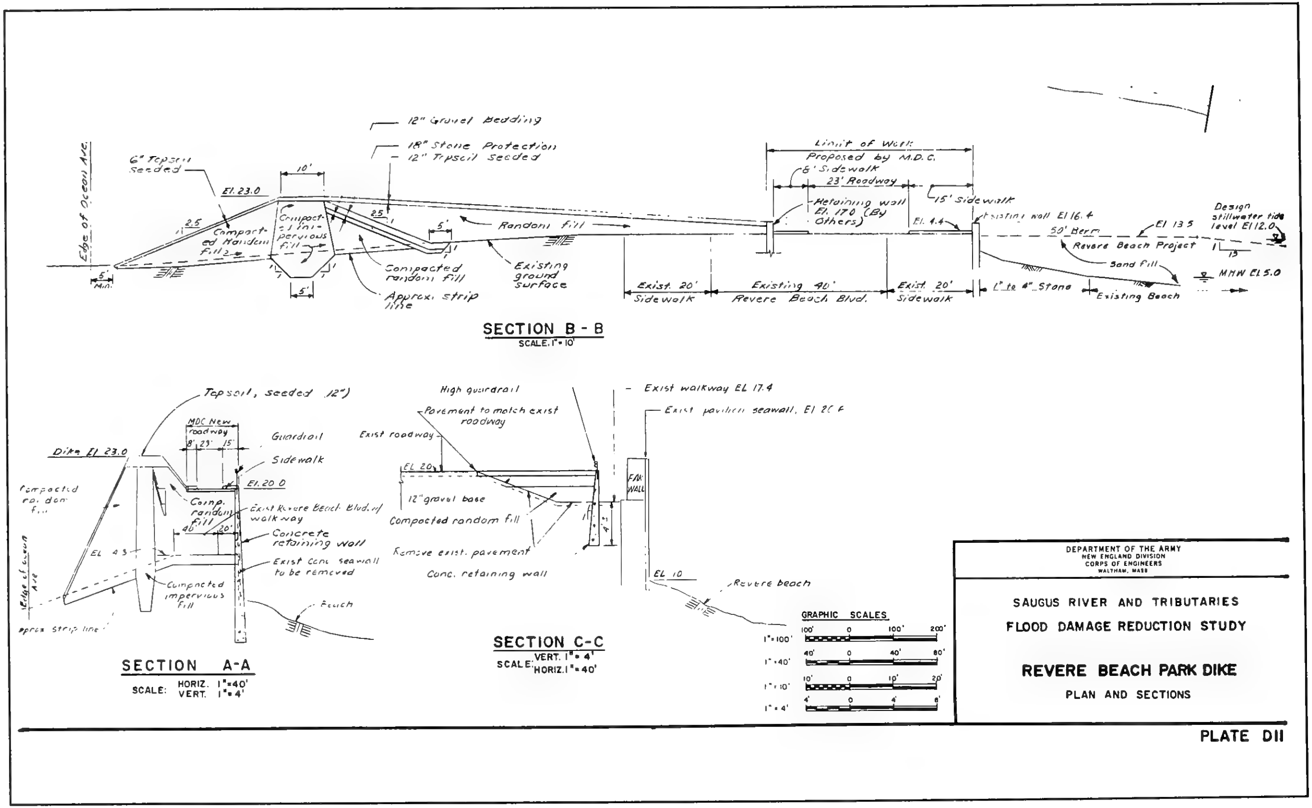

Figure 13. Portion of prototype plans for the proposed park dike

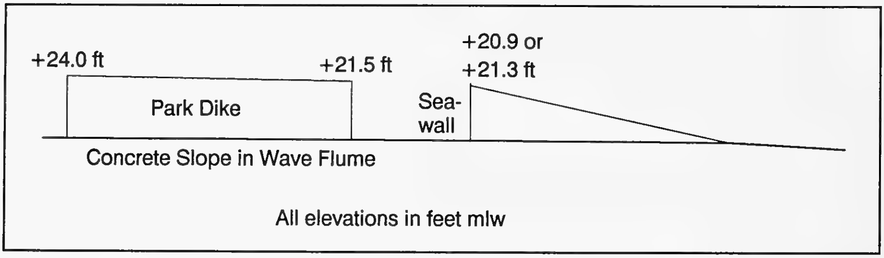

Figure 14. Cross section of model park dike

were conducted with beach elevations at the seawall of +14.4, +15.4, and +19.9 ft mlw to test the effects on overtopping rates of an eroded beach profile.

Test conditions and results

Tests were conducted for a prototype wave with H_{mo} = 12.7 ft, the largest wave height of the design storm. Prototype wave periods tested were T_p = 15.9 sec and 13.0 sec, corresponding to the peak periods of the SPN and 1978 storm, respectively. Water depth was +16.6 ft mlw, the greatest depth of the design storm. The beach profile tested was the 1991 post-storm profile with a maximum beach elevation of +20.5 ft mlw. With these test conditions, there was no measurable overtopping with the lowest crest elevation tested (+24.0 ft mlw); therefore, higher crest elevations were not tested under design storm conditions. A series of tests under less severe conditions had been planned, but were not conducted because of the lack of overtopping under the most extreme conditions of the SPN. Test conditions and measured overtopping rates are listed in Table 15.

Tests 1 and 2 were conducted with a seawall crest elevation of +21.3 ft mlw and peak wave periods of 15.9 and 13.0 sec, respectively. Tests 3 and 4 were conducted under the same wave conditions, but with the seawall crest lowered to +20.9 ft mlw. Minor overtopping was observed in all four tests, but the overtopping quantities were not sufficient to measure. Qualitatively, higher overtopping rates were observed with the higher seawall crest elevation and higher wave period.

The sheet metal beach in front of the seawall was removed for Tests 5 and 6 to determine effects of beach erosion on overtopping rates. With a beach elevation of +15.4 ft mlw, overtopping rates increased considerably, although the overtopping was still not measurable at the 13.0-sec peak wave period.

For Test 7, the swl was increased by 1 ft to +17.6 ft mlw to simulate possible sea level rise during the life of the structure. The additional foot

Table 15
Test Conditions and Overtopping Rates for Park Dike with 1991 Profile

Run No.	SWL ft mlw	Wave Height ft	Wave Period sec	Seawall Crest Elev. ft mlw	Dike Crest Elev. ft mlw	Beach Elev. ft mlw	Over-topping cfs/ft
1	16.6	12.7	15.9	21.3	24.0	20.5	[1]
2	16.6	12.7	13.0	21.3	24.0	20.5	[1]
3	16.6	12.7	15.9	20.9	24.0	20.5	[1]
4	16.6	12.7	13.0	20.9	24.0	20.5	[1]
5	16.6	12.7	15.9	20.9	24.0	15.4	0.0003
6	16.6	12.7	13.0	20.9	24.0	15.4	[1]
7	17.6	12.7	15.9	20.9	24.0	15.4	0.0068
8	16.6	12.7	15.9	25.9	[2]	15.4	0.0083
9	17.6	12.7	15.9	20.9	25.7	15.4	0.0011
10	17.6	12.7	15.9	20.9	25.7	20.0	0.0009
11	17.6	12.7	15.9	21.3	27.5	20.6	0.0003

[1] Overtopping too low to be measured.
[2] Overtopping of seawall, not park dike.

of depth greatly increased the overtopping rates to nearly 0.007 cfs/ft (prototype).

Test 8 increased the seawall crest elevation by 5 ft to +26.9 ft mlw to determine if an increase in seawall crest would prevent overtopping without the expense of the park dike. Overtopping was measured directly behind the seawall and averaged 0.008 cfs/ft. The additional foot of depth used in Test 7 was not used in Test 8.

Tests 9 through 11 brought the swl back up to +17.6 ft mlw. Tests 9 and 10 raised the crest of the park dike to +25.7 ft mlw and used beach elevations of +14.4 and +19.9 ft mlw, respectively. Test 11 raised the park dike crest elevation to +27.5 ft mlw, returned the seawall crest elevation to +21.3, and replaced the sheet metal slope in front of the seawall to a beach elevation of +20.6 ft mlw. The park dike was overtopped in all three tests.

Rubble-Mound Dike, 1991 Profile

Model construction

Modeling of the rubble-mound dike assumed the prototype would be constructed with an impermeable core covered by an underlayer and two layers of armor stone. The model rubble-mound dike was constructed of a piece of plywood for the impermeable core with crushed gravel retained by a No. 6 sieve glued to the board to simulate the underlayer. Crushed gravel passing a 3/4-in. sieve and retained by a 5/8-in. sieve was used for the armor stone. Average weight of the armor stones in the model was 0.022 lb (672 lb prototype).

The toe wall used for the park dike was used again for the rubble-mound dike but repositioned further back from the seawall on the assumption the mound would be built on the west side of the existing Revere Boulevard. Crest height of the impermeable core was set at +25.4 ft mlw. The rubble mound was constructed to the dike crest, and the area behind the crest was sealed to retain the overtopping (Figure 15).

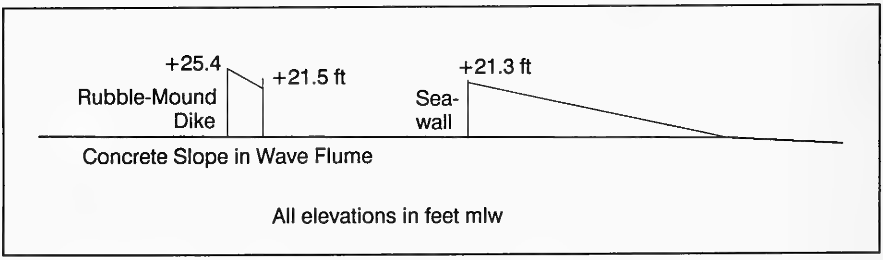

Figure 15. Cross section of rubble-mound dike

A seawall crest elevation of +21.3 ft mlw was used for all tests because it was observed in the park dike study that this crest elevation produced more overtopping than the lower crest elevation.

Test conditions and results

Generally, the same test conditions used in the park dike study were used for the rubble-mound dike. Still-water level was at +16.6 ft mlw, and beach elevation was set at +20.5 ft mlw. Test conditions and overtopping rates are listed in Table 16.

Tests 12 and 13 used a peak wave period of 13 sec and H_{mo}'s of 10.0 and 12.7 ft, respectively. There was no overtopping in either test.

Table 16
Test Conditions and Overtopping Rates for Rubble-mound Dike with 1991 Profile

Run No.	SWL ft mlw	Wave Height ft	Wave Period sec	Seawall Crest Elev. ft mlw	Dike Crest Elev. ft mlw	Beach Elev. ft mlw	Over-topping cfs/ft
12	16.6	10.0	13.0	21.3	25.4	20.5	0.0000
13	16.6	12.7	13.0	21.3	25.4	20.5	0.0000
14	16.6	12.7	15.9	21.3	25.4	20.5	0.0000
15	16.6	11.0	15.9	21.3	25.4	20.5	0.0000
16	17.6	12.7	15.9	21.3	25.4	20.5	0.0013
17	16.6	12.7	15.9	21.3	25.4	15.4	[1]
18	17.6	12.7	15.9	21.3	25.4	15.4	0.0005
19	16.6	12.7	15.9	21.3	25.4	17.8	[1]
20	17.6	12.7	15.9	21.3	25.4	17.8	[1]

[1] Overtopping too low to be measured.

Tests 14 and 15 used a peak wave period of 15.9 sec and H_{mo}'s of 12.7 and 11.0 ft, respectively. Again there was no overtopping.

Although there was no overtopping in Tests 12 through 15, there was some splashing over the rubble mound. The quantity of splashing was too small to measure. Wave runup approached the crest of the mound without flowing over the crest.

For Test 16, the swl was raised 1 ft to +17.6 ft, simulating possible sea level rise. Peak period was 15.9 sec and wave height was 12.7 ft. The high-water level produced an overtopping rate of 0.001 cfs/ft.

For Tests 17 and 18, the beach in front of the seawall was removed, leaving a beach elevation of +15.4 ft mlw. Still-water level was returned to +16.6 ft mlw for Test 17 and +17.6 ft for Test 18. Overtopping was observed in both tests but was not sufficient to measure at the lower water level (Test 17). Overtopping rate for Test 18 was 0.0005 cfs/ft.

For Tests 19 and 20, the beach was partially restored to an elevation of +17.7 ft mlw. The still-water level was set at +16.6 ft mlw for Test 19 and raised to +17.6 ft for Test 20. Overtopping was observed in both tests, but was not measurable.

There was no armor instability in any of the tests in this test series.

Park Dike, 1978 Profile

Model construction

The park dike model was constructed identically to the model with the 1991 profile, except that tests with the 1978 bathymetry were conducted in the 18-in. flume rather than the 3-ft flume. Crest elevation of the park dike was +24.0 ft mlw.

Test conditions and results

Table 17 lists conditions used to test the park dike. The swl's selected were the highest water levels predicted for the SPN and the highest swl observed during the 1978 storm. Similarly, wave periods were maximum wave periods for the SPN and the 1978 storm. Wave heights were selected as maximum wave height of the SPN plus lower wave heights to provide a range of overtopping values. See Table 4, Profile 2, for SPN conditions, and Table 2 for conditions in the 1978 storm.

The wave generator in the 18-in. flume was unable to produce the maximum wave conditions selected for testing. For test conditions where the wave generator was unable to produce the desired wave spectrum, the H_{mo} was incrementally decreased by 10 percent until conditions were within the limits of the wave generator. Water depths and wave periods were not changed. Table 17 lists wave heights that were used in the test series.

Tests 21 through 33 in Table 17 were tested with the same geometry of beach, seawall, and dike elevations. For the remaining tests (34 though 45), the swl was kept constant at +16.5 ft mlw and the same dike elevation of +24.0 ft mlw was maintained. Four of the test conditions in Table 17 (two wave heights at each of the two wave periods) were selected for each of the three remaining sets of tests. Tests 34 through 37 measured overtopping rates with the beach elevation raised to +13.1 ft mlw, and Tests 38 through 41 further increased the beach elevation to +16.8 ft mlw. For Tests 42 through 45, the beach was returned to the 1978 profile elevation of +9.3 ft mlw and the seawall crest elevation was reduced to +18.5 ft mlw (approximate elevation of Revere Boulevard in the vicinity of Profile 2). Tests 42 through 45 were meant to simulate conditions if the seawall were to fail.

Testing the park dike with the 1978 bathymetry produced very large overtopping quantities. At swl = +16.5 ft mlw and T_p = 15.9 sec, the smallest wave heights tested (H_{mo} = 8.9 ft, Test 21) caused sheets of water to completely overtop the structure, with some spray passing over the entire park dike while still in the air. Vertical spray exceeded the height of the flume, and a board was placed on top of the flume to retain the spray. These extreme cases of overtopping occurred when groups of large waves

Table 17
Test Conditions and Overtopping Rates for Park Dike with 1978 Profile

Run No.	SWL ft mlw	Wave Height ft	Wave Period sec	Seawall Crest Elev. ft mlw	Dike Crest Elev. ft mlw	Beach Elev. ft mlw	Over-topping cfs/ft
21	16.6	8.9	15.9	21.3	24.0	9.3	0.0855
22	16.6	11.0	15.9	21.3	24.0	9.3	0.0960
23	16.6	10.0	15.9	21.3	24.0	9.3	0.0844
24	16.6	11.4	13.0	21.3	24.0	9.3	0.0836
25	16.6	11.0	13.0	21.3	24.0	9.3	0.0713
26	16.6	10.0	13.0	21.3	24.0	9.3	0.0642
27	14.8	9.6	15.9	21.3	24.0	9.3	0.0015
28	14.8	8.4	15.9	21.3	24.0	9.3	0.0041
29	14.8	7.2	15.9	21.3	24.0	9.3	0.0002
30	14.8	9.9	13.0	21.3	24.0	9.3	0.0017
31	14.8	8.6	13.0	21.3	24.0	9.3	0.0026
32	14.8	7.7	13.0	21.3	24.0	9.3	0.0013
33	17.6	8.9	15.9	21.3	24.0	9.3	0.2574
34	16.6	8.9	15.9	21.3	24.0	13.1	0.0220
35	16.6	10.0	15.9	21.3	24.0	13.1	0.0437
36	16.6	11.4	13.0	21.3	24.0	13.1	0.0278
37	16.6	10.0	13.0	21.3	24.0	13.1	0.0207
38	16.6	8.9	15.9	21.3	24.0	16.8	0.0121
39	16.6	10.0	15.9	21.3	24.0	16.8	0.0162
40	16.6	11.4	13.0	21.3	24.0	16.8	0.0058
41	16.6	10.0	13.0	21.3	24.0	16.8	0.0000
42	16.6	8.9	15.9	18.5	24.0	9.3	0.0719
43	16.6	10.0	15.9	18.5	24.0	9.3	0.0769
44	16.6	11.4	13.0	18.5	24.0	9.3	0.0612
45	16.6	10.0	13.0	18.5	24.0	9.3	0.0644

prevented rundown on the slope and produced a hydraulic head between the park dike and the seawall.

Test 22 increased the wave height to 11 ft and produced greater overtopping. Test 23 used the same conditions as Tests 21 and 22, except for a wave height of 10.0 ft. Although the wave height in Test 23 was 1.1 ft greater than in Test 21, the measured overtopping was less by about 1 percent. This slight discrepancy could be caused by the random nature of the wave trains being used or by inaccuracies in the collection and measurement. However, Tests 27 through 29 differed only in wave height, and Test 27 with a wave height of 9.6 ft had a low overtopping rate relative to Tests 28 and 29. Similarly, Tests 30 through 32 differed only in wave height, and Test 30 with a wave height of 9.9 ft had a low overtopping rate relative to Tests 31 and 32. In each of these sets of tests, wave heights around 9 to 10 ft were seen to produce surprisingly low overtopping. This trend of low overtopping rates was observed only with a beach elevation of +9.3 ft mlw and was not observed in Tests 34 through 41, which used a higher beach elevation, or in Tests 42 through 45, which used a lower seawall elevation.

As expected, reducing the swl to +14.8 ft mlw in Tests 28 through 32 greatly reduced overtopping rates, while increasing the swl to +17.6 ft in Test 33 nearly inundated the structure.

Raising the beach elevation in front of the seawall to +13.1 ft mlw in Tests 34 through 37 decreased overtopping rates, and further increasing the beach elevation to +16.8 ft in Tests 38 through 41 further decreased overtopping rates. The only test of the park dike with the 1978 profile that did not produce overtopping was Test 41 with the beach elevation at +16.8 ft mlw.

Tests 42 through 45 returned the beach profile to the conditions of the 1978 survey (+9.3 ft mlw) and reduced the seawall elevation 2.8 ft to +18.5 ft mlw. Overtopping rates were less than under the same conditions but with the seawall intact (Tests 21, 23, 24, and 26) for Tests 42 through 44, and showed little change in Test 45. This was consistent with the finding reported above in the tests of the park dike with the 1991 profile; i.e., higher overtopping rates were obtained with the higher seawall elevation.

Rubble-Mound Dike, 1978 Profile

Model construction

The rubble-mound dike model was constructed in the same manner as the 1991 profile, but a smaller armor stone was used. Although specific tests for stability were not conducted, there was no movement of armor stone observed on tests with the 1991 profile. The armor stone was therefore reduced to crushed gravel passing a 5/8-in. sieve and retained by a 1/2-in. sieve. The model armor stone had an average weight of 0.011 lb per stone (336 lb prototype).

Test conditions and results

Three sets of four tests each were conducted with each set consisting of one wave height at each of two wave periods at each of two swl's. Test conditions were the highest obtainable wave height at each of the wave periods and swl's listed in Table 17, with the exception of swl = +17.6 ft mlw, which exceeds the design storm conditions and was not tested in this series. Tests 46 through 49 were conducted with the beach elevation at +9.3 ft mlw (1978 survey), Tests 50 through 53 repeated the wave conditions but with the beach elevation raised to +14.3 ft mlw, and Tests 54 through 57 raised the beach elevation to +16.7 ft mlw. Test conditions, measured overtopping rates, and number of armor stones displaced are listed in Table 18.

Although the rubble-mound dike never approached a failure condition, with failure defined as having the underlayer exposed, armor stones were displaced during several of the tests. Displaced armor stones were replaced on the structure only after each set of four tests.

Tests 46 through 49 (beach elevation as measured in the 1978 survey) all produced overtopping. Some armor stones were displaced, with

Table 18
Test Conditions, Overtopping Rates, and Armor Stone Displacement for Rubble-mound Dike with 1978 Profile

Run No.	SWL ft mlw	Wave Height ft	Wave Period sec	Seawall Crest Elev. ft mlw	Dike Crest Elev. ft mlw	Beach Elev. ft mlw	Over-topping cfs/ft	Armor Stone Displacement	
								Seaward	Shoreward
46	16.6	11.0	15.9	21.3	25.4	9.3	0.0618	15	18
47	16.6	11.4	13.0	21.3	25.4	9.3	0.0446	9	0
48	14.8	9.6	15.9	21.3	25.4	9.3	0.0026	0	0
49	14.8	9.9	13.0	21.3	25.4	9.3	0.0024	3	0
50	14.8	9.9	13.0	21.3	25.4	14.3	0.0000	0	0
51	14.8	9.6	15.9	21.3	25.4	14.3	0.0000	0	0
52	16.6	11.4	13.0	21.3	25.4	14.3	0.0166	6	8
53	16.6	11.0	15.9	21.3	25.4	14.3	0.0067	2	0
54	16.6	11.0	15.9	21.3	25.4	16.7	0.0054	2	0
55	16.6	11.4	13.0	21.3	25.4	16.7	0.0045	0	0
56	14.8	9.6	15.9	21.3	25.4	16.7	0.0000	0	0
57	14.8	9.9	13.0	21.3	25.4	16.7	0.0000	0	0

15 stones moved to in front of the toe wall and 18 stones carried over the crest of the dike during Test 46, 9 stones displaced to in front of the toe wall in Test 47, and 3 stones displaced to seaward of the toe wall in Test 49. Displaced stones were not replaced until after Test 49.

For Tests 50 through 53, the beach elevation in front of the seawall was raised to +14.3 ft mlw. There was no overtopping in Test 50 and only a very small and unmeasurable overtopping from one wave in Test 51. Neither Test 50 nor 51 had any armor stones displaced. Tests 52 and 53 had measurable overtopping, with six armor stones displaced seaward and eight armor stones displaced shoreward during Test 52, and two armor stones displaced seaward in Test 53.

Tests 54 through 57 raised the beach elevation in front of the seawall to +16.7 ft mlw. Minor overtopping was observed during Tests 54 and 55 with two armor stones displaced seaward in Test 54. There was no overtopping and no armor stone displacement in Tests 56 and 57.

Armor Unit Stability

Stability of armor units on the rubble-mound dike was not specifically tested, but the following information may be of value for design purposes.

As reported above, there was no armor stone displacement using stones with an average weight of 0.022 lb and bathymetry from the 1991 survey. Armor stone displacement during tests with the 1978 bathymetry and armor stones averaging 0.011 lb are given in Table 18. Armor stones used in the tests were a crushed dolomite with a unit weight of 165 pcf. Based on relationships defined by Froude's model law (see Chapter 2, "Test Facility"), the following transference equation is derived to determine prototype stone weights.

$$\frac{(W_a)_m}{(W_a)_p} = \frac{(\gamma_a)_m}{(\gamma_a)_p} \left(\frac{L_m}{L_p}\right)^3 \left[\frac{(S_a)_p - 1}{(S_a)_m - 1}\right]^3$$

where

W_a = weight of an individual stone, lb

subscripts m,p = model and prototype values, respectively

γ_a = specific weight of an individual stone, pcf

L_m/L_p = linear scale of the model

S_a = specific gravity of an individual stone relative to the water in which the breakwater is constructed, i.e., $S_a = \gamma_a/\gamma_w$

γ_w = specific weight of water, pcf

Assuming a specific weight of seawater of 64.0 pcf and fresh water of 62.4 pcf, assuming a specific weight of 165 pcf for both model and prototype stone, and using a model scale of 1:30, average weights of armor stone used in the models correspond to average prototype weights of 672 lb and 336 lb for tests conducted with the 1991 profile and 1978 profile, respectively.

6 Revere Dike Discussion

The proposed park dike with the crest lowered to provide only a 1.5-ft rise from toe wall to crest was found to be sufficient to prevent nearly all overtopping during the design storm event using the post-storm 1991 beach profile. Revere Boulevard, of course, would be completely flooded. In the model, waves overtopping the seawall and crossing Revere Boulevard would flow part way up the park dike in a solid sheet of water across the width of the flume. As the runup decreased, the sheet of water would be reduced to a few "fingers" or thin streams of water that flowed much further up the slope of the dike. Under the most severe conditions of the SPN, most sheet flow did not extend more than one-third to one-half the distance to the crest of the dike before separating into a few "fingers." All overtopping observed with the park dike under design storm conditions with the 1991 bathymetry occurred when one of the "fingers" reached the crest of the dike. At no time did the solid sheet of runup reach the crest. It is anticipated that prototype runup on a park dike covered with vegetation and paths would be less than observed in the wave flume. If the profile in front of the seawall is maintained at a bathymetry similar to the 1991 survey, the park dike with a crest elevation of +24.0 ft mlw should be adequate to prevent nearly all overtopping during the design storm event. It should be recognized, however, that in any random sea event there is a possibility of an event occurring that exceeds the conditions tested in the physical model.

Decreasing the elevation of the seawall decreased the rate of overtopping over the park dike. With the toe wall maintained at a constant elevation, decreasing the freeboard of the seawall increased the freeboard of the toe wall over the seawall by the same amount, increasing the effectiveness of the toe wall. Although there was more overtopping of the lower seawall, the increased effectiveness of the toe wall resulted in less water overtopping the dike. Tests conducted to determine effects of a failure of the seawall to the elevation of the roadway found that overtopping rates were lower than with the seawall in place.

Wave breaking occurred either on the slope in front of the seawall or across the seawall onto Revere Boulevard. By the time wave action crossed the toe wall onto the park dike, most of the turbulence had dissipated, and flow on the dike appeared to be predominantly laminar. If the park dike is covered with dirt and vegetation, scour can be expected during peak levels of the design hydrograph. Extensive scouring is not expected, however, due to the dissipation of the turbulence and the short duration of the hydrograph peaks. It is doubtful that the rockfill inside the park dike specified in the plans is necessary.

A small rubble mound with an impervious core was found to be effective in preventing overtopping when tested with the 1991 profile. The roughness of the stone structure quickly halted the runup, and a much smaller structure than the park dike was found to be sufficient. With a crest elevation of +25.4 ft mlw, there was no overtopping during design storm conditions, with the exception of a minor quantity of splashing. However, runup was observed to approach the mound's crest, and overtopping would have occurred at a lower mound crest elevation. Overtopping was observed on tests conducted to simulate beach erosion in front of the seawall, increased swl from sea level rise, or on tests with the 1978 profile.

Displacement of armor stones occurred with stones averaging 336 lb (prototype) during tests with the 1978 profile. It should be noted that armor stone displacement is common with new construction, and typically decreases as the stones become seated by wave action. Although the number of armor units displaced decreased during each successive set of tests with the 1978 profile and the rubble-mound dike, the amount of wave action on the dike was less during each successive set of tests due to increases in the beach elevation. Because wave action on the dike with the 1991 profile was less than with the 1978 profile, and assuming that stones would be seated during storms of less severity than the design event, the 336-lb stones are probably sufficient if the 1991 beach profile is maintained.

If the beach profile returns to a bathymetry similar to the 1978 profile, both park dike and rubble-mound dike will be overtopped during the design storm event and under less extreme conditions. Because sea conditions varied for the various tests conducted under this research effort, it is difficult to compare overtopping rates for the different profiles and structure options at Revere Beach. However, the following comparison, based on conditions at the peak of the SPN, may be instructive.

Table 19 lists several tests of Profile 2 tested at the peak of the SPN with an swl of +16.6 ft mlw and a wave period of 15.9 sec. Both 1978 and 1991 profiles are included, as are both park dike and rubble-mound dike conditions, as well as overtopping rates without a dike.

By far the highest overtopping rate was found with the 1978 profile and no dike. With the addition of the beach fill (1991 profile), the overtopping rate was reduced by about 65 percent even with a wave height that was half again as high as that conducted on the 1978 profile. The addition of either the park dike or rubble-mound dike, with the 1991 profile, reduced the overtopping to nearly zero. The physical model did not include erosion of the 1991 beach profile during the SPN storm. Overtopping rates may be expected to range between those measured with the 1991 and

Table 19
Comparison of Overtopping Rates at Peak of SPN

Table in which data are listed	Year of Survey	Dike	SWL	Wave Height	Wave Period	Over-topping Rate
12	1978	None	16.6	8.86	15.9	1.1980
8	1991	None	16.6	12.65	15.9	0.4156
15	1991	Park	16.6	12.70	15.9	[1]
16	1991	Rubble-Mound	16.6	12.70	15.9	0.0000
17	1978	Park	16.6	11.00	15.9	0.0960
18	1978	Rubble-Mound	16.6	11.00	15.9	0.06180

[1] Overtopping rate too small to be measured.

1978 profiles because erosion of the 1991 beach profile during the SPN storm can be expected.

If the beach erodes back to the 1978 profile, overtopping of either dike will occur. The last two entries in Table 19 give overtopping rates for the park dike and rubble-mound dike with the 1978 profile. The dikes greatly reduced the overtopping rate compared to conditions without any dike, although the overtopping rates with either dike may still be unacceptable if the beach is eroded to the 1978 profile.

As a qualitative reference, Fukuda, Uno, and Irie (1974) measured and filmed waves overtopping a seawall fronted by a concrete revetment during severe storms. The films were then viewed by a panel of coastal experts who estimated the degree of danger posed by the overtopping. Averaging the results of the panel, it was determined that at a location 10 ft behind the structure, overtopping rates greater than 0.0002 cfs/ft would prohibit a vehicle from driving past at high speed, damage to a house could be expected at an overtopping rate of 0.0007 cfs/ft, and overtopping rates greater than 0.002 cfs/ft would be dangerous for a walking person. These overtopping rates assumed an average over several hundred waves and could be increased by a factor of 10 for a location 30 ft behind the structure. For protection of a relatively densely populated coastal area, Goda (1985) reports an overtopping rate of 0.1 cfs/ft as an adopted guideline in port areas in Japan. These overtopping rates assume an average over several hundred waves (Goda 1985).

Based on the overtopping rates given above from Fukuda, Uno, and Irie (1974), overtopping rates with either dike may be hazardous if the beach erodes to the 1978 condition. However, conditions listed in Table 19 occurred only at the peak of the SPN hydrograph and the dikes eliminated 95 to 98 percent of the overtopping compared to the 1978 condition. Lower swl's caused substantially less overtopping (see Tables 17 and 18).

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Physical model studies were conducted in 2-D wave flumes to determine overtopping rates for existing and proposed structures along Revere Beach, Massachusetts, during design storm events. Results of the physical model studies were used in regression analysis to develop simple nondimensional equations relating overtopping rates to incident wave conditions and structure design along various reaches of the beach. Total volume overtopping in the physical model was compared to prototype data for a known storm event to verify the model. Overtopping rates were then determined for the existing seawall using bathymetries surveyed both before and after a 1991 beachfill project. The effectiveness of a proposed dike shoreward of the seawall was also measured in the physical model.

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